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Power Division

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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VOLUME 82

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

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2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, 3/16-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller in the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper within a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

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8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

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Proceedings of the American Society of Civil Engineers

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DISCUSSION
(Proc. Paper 1180)

Discussion of
"THE HIGH-SYPHON CIRCULATING WATER SYSTEM—MERAMEC PLANT"

by Charles E. Buettner and Paul A. Pickel
(Proc. Paper 740)

CHARLES E. BUETTNER,[1] A.M. ASCE and PAUL A. PICKEL.[2]—The writers are gratified by the favorable comment the paper evoked from Messrs. Richards and Rudulph.

The facts that Mr. Richards presented, from actual tests, regarding water column separation in circulating water systems and the effects of this phenomena are a substantial contribution to a field of knowledge wherein little has been published.

Mr. Rudulph emphasized the importance of the circulating system in modern thermal power plants, the massiveness of certain aspects of such systems and the complexity of the problem of relating these aspects to one and another effectively.

It is hoped that the publication of this paper and the subsequent discussion will result in further investigation and publication in this field, by others.

1. Mechanical Engr., Union Electric Co. of Mo., St. Louis, Mo.
2. Project Engr., Union Electric Co. of Mo., St. Louis, Mo.

Discussion of
THE HIGH-SYPHON CIRCULATING WATER SYSTEM—A [illegible] PLANT

by Charles E. Buettner and Paul A. Peckel
(Proc. Paper 1401)

CHARLES E. BUETTNER,[1] A.M. ASCE and PAUL A. PECKEL,[2]—The writers are gratified by the favorable comment the paper evoked from Messrs. Richards and Rudolph.

The facts that Mr. Rudolph presented, from actual tests, regarding water column separation in circulating water systems and the effects of this phenomena has a substantial contribution to a field of knowledge wherein little has been published.

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It is hoped that the publication of this paper and the subsequent discussion will result in further investigation and publication in this field, by others.

1. Mechanical Engr., Union Electric Co. of Mo., St. Louis, Mo.
2. Project Engr., Union Electric Co. of Mo., St. Louis, Mo.

Discussion of
"ARCH DAMS: DESIGN AND OBSERVATION OF ARCH DAMS IN PORTUGAL"

by M. Rocha, J. Laginha Serafim, and A. F. da Silveira
(Proc. Paper 997)

CORRECTIONS.—On page 997-37, on line 3, the equation 57 - 6 = 51 $kgcm^{-2}$ should be changed to 57 + 6 = 63 $kgcm^{-2}$. On this same page, on line 5, the equation 88 - 14 = 64 $kgcm^{-2}$ should be changed to 88 - 24 = 64 $kgcm^{-2}$. In Fig. 7, Studies I and II, Upstream Face, at El. 640 on the left side the value -23 should be changed to +23. In Fig. 11, Study I, Upstream Face, at El. 210 the stress -122 should have the direction of the arrow reversed. In Fig. 15, Study I, Upstream Face, Lateral Cantilever, at El. 230 the term +8 should be changed to -8. In Fig. 15, Study II, Upstream Face, Crown Cantilever, at El. 180 the term -38 should be changed to +38. In Fig. 15, Study V, Downstream Face, Lateral Cantilever, at El. 250 the term -8 should be changed to +8. In Fig. 20, Stress at Downstream Face, Crown Cantilever, at El. 250 the term -17 should be changed to +17. In Fig. 20, Temperature Variations, Crown Cantilever, at El. 250 the term +2, 6 should be changed to +1, 6.

ROBERT E. GLOVER,[1] M. ASCE.—In this paper the authors present a valuable series of correlations among the results of analytical studies and model studies for arch dams and observations of the prototype structures under load. They find a very favorable comparison between the model test results and prototype stresses after the prototype observations have been corrected for stresses, due to thermal changes, which were not present in the models. Correlations with incomplete trial load analyses, based upon a radial adjustment only, showed arch stresses from ten to twenty percent higher than those obtained from the models and gave cantilever tensile stresses at the base which were much higher than those found in the models. A complete trial load study for the important Cabril Dam appears to have given a much better correlation.

These relationships are as they should be since a radial adjustment alone does not fulfill the basic requirements for a proper solution of the stress and strain distribution in a structure. These requirements are (1) that every element of the structure should be in equilibrium under the stresses and forces which act upon it., (2) that every element of the structure must deform in such a way that it continues to fit with its neighbors on all sides as the structure passes from the unstrained state to the strained state and (3) that the appropriate boundary conditions must be met. The Kirchhoff uniqueness theorem[2] contains a proof that there is only one stress distribution capable of meeting these requirements. A radial adjustment alone meets

---
1. Research Engr., U. S. Bureau of Reclamation, Denver, Colo.
2. The Mathematical Theory of Elasticity, by A. E. H. Love., Fourth Edition-Cambridge University Press-1927-Paragraph 118.

requirements (1) and (3) but leaves the important continuity requirement (2) unsatisfied. The displacements of a prism, common to an arch and cantilever element where they cross, as given by the arch and cantilever computations, will illustrate this point. For all points between the arch crown and abutment the arch computation will indicate that the prism will have been displaced toward the abutment and rotated about a vertical axis, whereas the cantilever computation will imply only a displacement radially downstream without this kind of tangential displacement or rotation. The cantilever computation will, conversely, show a rotation about a horizontal axis which is not shared by the arch displacements. A radial adjustment only, therefore, leaves every element of the dam occupying two positions. This is a situation which is obviously faulty. If this distribution is tested against the Kirchhoff requirements it can only be concluded that it is not the unique stress distribution which must prevail. It should be expected therefore that a trial load study, based upon a radial adjustment only, will not agree with the results of a model test. This expectation is confirmed by many such comparisons.

It is the purpose of the tangential and twist adjustments to eliminate the tangential and rotational displacement discrepancies left by the radial adjustment. These adjustments bring into evidence the tangential and vertical shear forces which act upon the top, bottom and sides of the element. By so doing the important elements of strength provided by shearing and twist resistances of the arch and cantilever elements are accounted for. The effect of adding these two adjustments is generally to decrease the computed deflections and to lower the computed stresses. A trial load analysis including carefully made radial, tangential and twist adjustments will generally agree closely with the results of model tests. The tangential and twist effects contribute most effectively to the strength of arch dams constructed in wide sites. They may be nearly absent in dams built in very narrow canyons.

A complete agreement between the results of a model test, made within the elastic range, and an analysis satisfying the equilibrium continuity and boundary condition requirements is to be expected. There can be but one solution to the elastic equations and when a computer has satisfied these three requirements he has that solution. This will be identical with the model distribution also since the conditions it must meet are precisely those enumerated.

It is important to arch dam designers that the analytical and experimental techniques they use for design be brought into complete accord. This can be done if computers will bring their analysis into agreement by trial-load procedures or otherwise, with all three of the requirements described above.

It may be noted that correlation with test data makes more exacting demands upon a computer than does the task of designing a dam to carry a specified loading with a given factor of safety. If the design computations neglect certain elements of strength, the net result will generally be a structure that is somewhat overdesigned but which may, nevertheless, be expected to serve well. In order to correlate with test data, however, the computer must find the unique solution and this is a much more exacting task.

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DESIGN OF THE EKLUTNA PROJECT, ALASKA

Frank B. Cook,* M. ASCE and David L. Goodman,** A.M. ASCE
(Proc. Paper 1132)

SYNOPSIS

This paper describes the design of the Eklutna Project, a hydroelectric power development of the Bureau of Reclamation near Anchorage, Alaska. Included in the paper are a description of the overall purpose and function of the project and a detailed summary of the design of the principal power facilities.

INTRODUCTION

The Eklutna Project is a 30,000-kilowatt hydroelectric power development designed and constructed by the Bureau of Reclamation, United States Department of the Interior, to bring urgently needed electric power to the rapidly expanding area at Anchorage, Alaska. The project, the first major development by the Bureau of Reclamation outside the continental United States, was constructed during the four-year period between 1951 and 1955. The first of the Eklutna Powerplant's two 15,000-kw generating units was placed in commercial operation on January 6, 1955; the second unit "went on the line" on April 1, 1955. Average annual firm energy provided by the plant is estimated to total 140 million kilowatt-hours. It is estimated that nonfirm energy, available seasonally in summer and early fall, will be approximately 20 million kilowatt-hours in the average year.

The Eklutna Powerplant (Figure 1) is on the Glenn Highway between Anchorage and Palmer, about 35 miles northeast of Anchorage. Transmission lines, operating at 115 kilovolts and totaling 41 miles in length, have been constructed to extend from the powerplant north to Palmer and south to

Note: Discussion open until May 1, 1957. Paper 1132 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 6, December, 1956.

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** Engr., Div. of Administrative Services, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

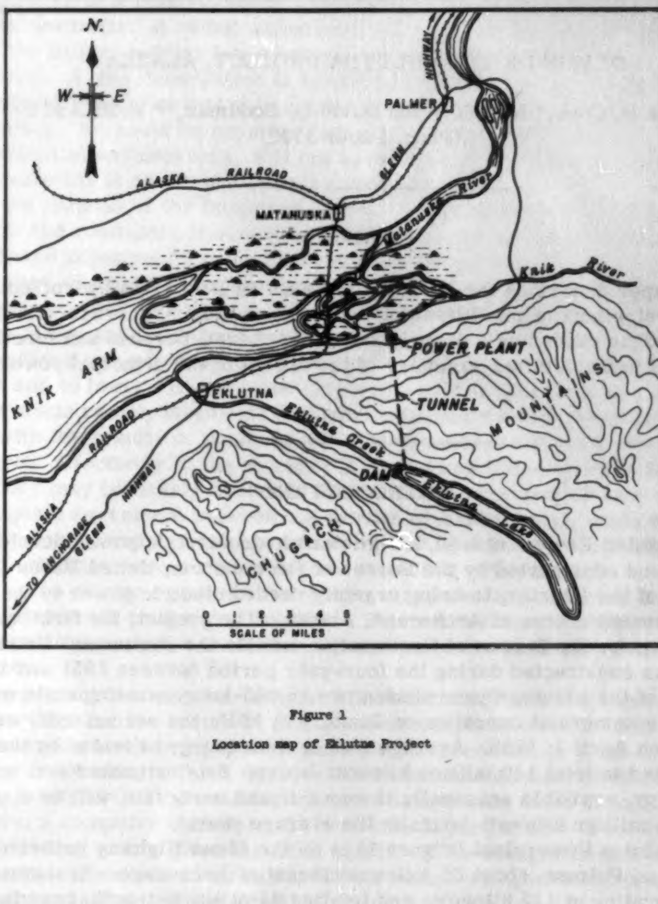


Figure 1

Location map of Eklutna Project

Anchorage. In addition, some existing lower voltage lines acquired with an older small hydroelectric plant (Little Eklutna) from the City of Anchorage have been consolidated in the system. Rural Electrification Administration cooperatives provide the distribution systems and facilities to communities outside of Anchorage. The new 115-kv main transmission system was adopted after studies of immediate and possible ultimate developments, which indicated this voltage level as the most desirable.

Development of the Eklutna Project is a major step toward realization of an enlarged regional economy, and the first step in major hydroelectric development in Alaska. The project is the first part of a comprehensive plan for conservation and development of water resources in this promising railbelt which extends from Seward through Anchorage and the Matanuska Valley and over the Alaska range towards Fairbanks. Construction of the main transmission lines for the Eklutna Project at 115-kv capacity will permit future expansion of the power system.

In the discussion that follows a description of the Eklutna Project is given and the significant details of design of the powerplant and its companion facilities are summarized.

Eklutna Project

The Eklutna Project was authorized by the Congress of the United States on July 31, 1950, to "encourage and promote the economic development of the Territory of Alaska, to foster the establishment of essential industries in said Territory, and to further the self sufficiency of national defense installations located therein." As authorized by the Congress, the primary function of the project is the production of electrical energy—energy which is urgently needed for civilian, military and industrial uses, and for meeting normal growth in domestic loads in the vicinity of the project.

Prior to construction of the project a serious power shortage prevailed in the Anchorage area. The area was dependent for its power supply upon a combination of small steam, hydroelectric, and diesel plants. Some of these plants were inadequate and in poor operating condition. Some military plants were also installed with principal capacity in steam-electric units. The lack of any margin of supply and high power rates greatly handicapped industrial and rural development, and fuel costs are very high in comparison to such costs in the United States. During World War II the need for power became increasingly critical. Several large defense establishments were constructed, among them Fort Richardson and Elmendorf Air Base near Anchorage. The wartime surge of construction and the postwar expansion of the city and its environs caused frequent electric light and power "brownouts." The development of the Eklutna Project has assisted in reducing the power shortage and has provided a stable and relatively low-cost power supply with which the previously existing and new fuel electric power facilities are being integrated.

The Eklutna project area includes the Willow Creek mining district on the north, the Matanuska Valley on the east, and the city of Anchorage and environs to Turnagain Arm on the south. Cook Inlet, a branch of the Gulf of Alaska, lies to the west. The area is a northern reach of the Pacific Mountain system, the parallel ranges of which enter Alaska through British Columbia, Canada, and embodies two large flats—a valley floor and a coastal plain. The Cook Inlet and the Chugach Mountains almost isolate these areas

from each other; they are connected by a narrow strip of land bordered by a branch of the inlet—Knik Arm, a tidal estuary—and the mountains.

Anchorage lies on a low bluff overlooking Cook Inlet, and is bounded on the north, south, and west by arms of the sea. To the east is a low plain extending to the Chugach Mountains. This enclosed area comprises about 75 square miles. Matanuska Valley, through which run the Matanuska and Knik Rivers, is roughly 50 by 16 miles, and is almost surrounded by the Alaska, Talkeetna, and Chugach Ranges.

Trending northwest from the Chugach Mountains to Knik Arm is the Eklutna Creek which descends through a steep-sided, troughlike, glaciated valley about 27 miles long. Rugged peaks up to 8,200 feet in elevation rise sharply above short valleys tributary to the creek.

At the head of Eklutna Creek is an alpine glacier, about 7 miles long. Its width tapers gradually from about 2 miles at elevation 4800 feet to several hundred feet where its snout lies at 1,000 feet. About 4 miles downstream from the snout of the glacier the creek empties into Eklutna Lake. The lake, formed when the glacier melted and its front receded to leave a natural dam across the creek, is 7 miles long, 0.7 mile wide, and 200 feet deep. The formation of this natural reservoir, 868 feet above sea level, and its diversion to the powerplant make possible the power development of the Eklutna Project.

The entire construction cost of the Eklutna Project will be repaid from power revenues. Under the Congressional authorization, the capital investment of the project, as determined by the Federal Power Commission, is to be amortized over a 50-year period and interest is to be charged on the unamortized balance of the full capital investment at a rate of 2-1/2 percent per year. The authorization states that: "Electric power and energy generated at the Eklutna Project, except that portion required in the operation of such project, shall be disposed of in such a manner as to encourage the most widespread use thereof at the lowest possible rates to consumers consistent with sound business principles * * * . Preference in the sale of such power and energy shall be given to all public bodies and cooperatives on the same terms, and to Federal agencies." The estimated average cost of firm power produced at this hydro plant is 11 mills per kilowatt-hour on the basis of full amortization of cost in less than 50 years with interest at 2-1/2 percent.

On November 23, 1953, a contract was completed with the city of Anchorage, providing for delivery of 16,000 kilowatts (maximum) for firm power. On November 1, 1954, a contract was completed with the Matanuska Electric Association of Palmer which provides for 5,000 kilowatts (maximum) contract rate of delivery for firm power. A third contract for the remaining 9,000 kilowatts is held with the Chugach Electric Association.

Design of Eklutna Powerplant and Related Features

In brief, the flow of water on the project for power production is as follows: Water is diverted from Eklutna Lake through the 4-1/2-mile Eklutna Tunnel under Goat Mountain (of the Chugach range) terminating at a surge tank, and thence through an underground penstock, about 1/4 mile long, to the Eklutna Powerplant. Water discharged from the powerplant's turbines passes through a conduit under the Glenn Highway to a 2,000-foot long open tailrace channel and into the Knik River which connects with Knik Arm. (See Figures

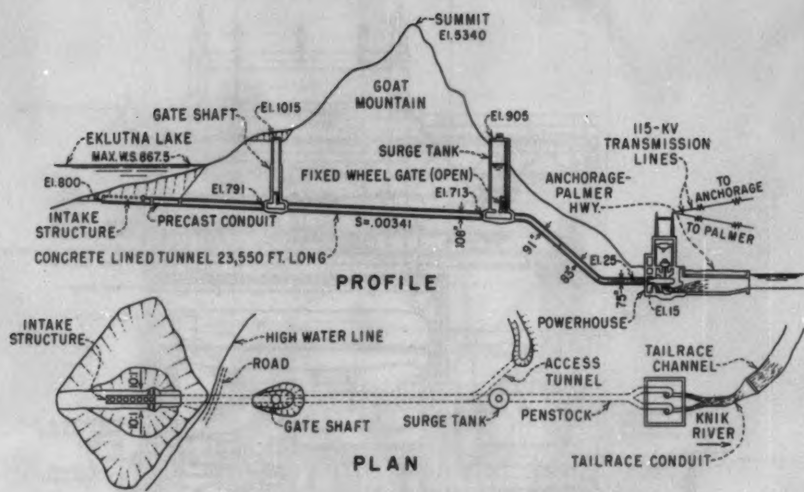
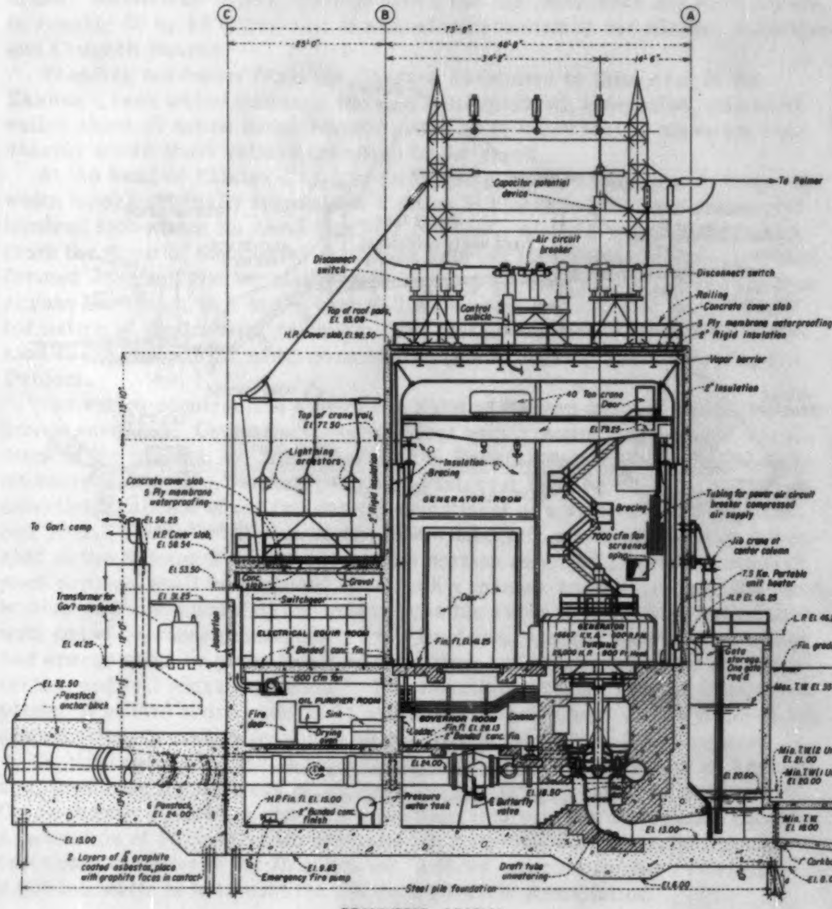


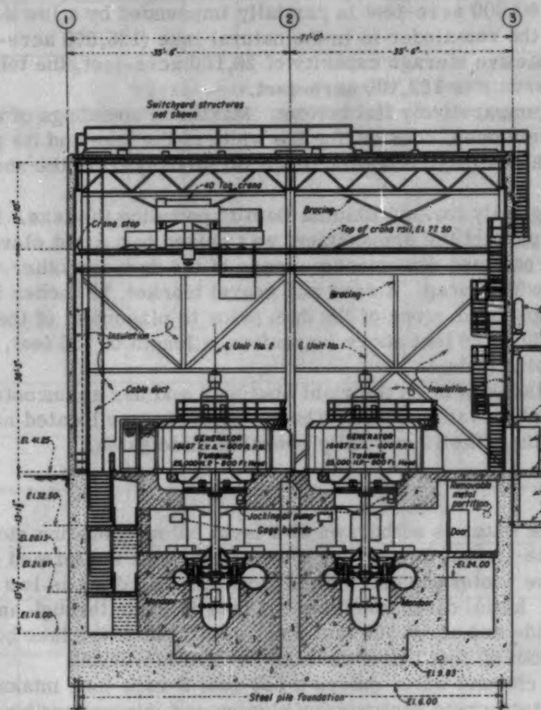
Figure 2

Schematic drawing of plan and profile of major structures of the Eklutna Project.



TRANSVERSE SECTION

SCALE OF FEET



LONGITUDINAL SECTION

SCALE OF FEET

1 and 2) Design of these hydraulic and power features and their companion facilities is described in that sequence of flow in the following summaries.

Eklutna Dam and Lake

The Eklutna Creek drainage basin embraces 172 square miles of which 119 square miles are tributary to Eklutna Lake. The water supply for the project comes from that portion of the creek runoff tributary to Eklutna Lake. Usable storage of 160,000 acre-feet is partially impounded by a low dam (24,000 acre-feet); the remainder is in the natural lake (136,000 acre-feet). Together with an inactive storage capacity of 22,100 acre-feet, the total capacity of the reservoir is 182,100 acre-feet.

The lake has a comparatively flat bottom. Maximum soundings of about 200 feet are relatively deep, considering the width of the lake and its geologic origin. Except at the outlet, the bottom drops off sharply from the shore before leveling off.

Glacial drift originally formed Eklutna Dam impounding the lake. During construction of the project the embankment was raised to a crest elevation of 875 feet. The upstream and downstream slopes of the dam and other areas were then covered with riprap. A sand and gravel blanket, 12 inches thick, was placed on the upstream slope of the dam prior to placement of the riprap. The dam has a height of 26 feet above foundation, a length of 555 feet, and a volume of 5,000 cubic yards.

The spillway of the dam is on the right abutment and has an uncontrolled crest 150 feet long at elevation 867.5. The outlet works are located in the river channel and comprise 19 manually operated slide gates.

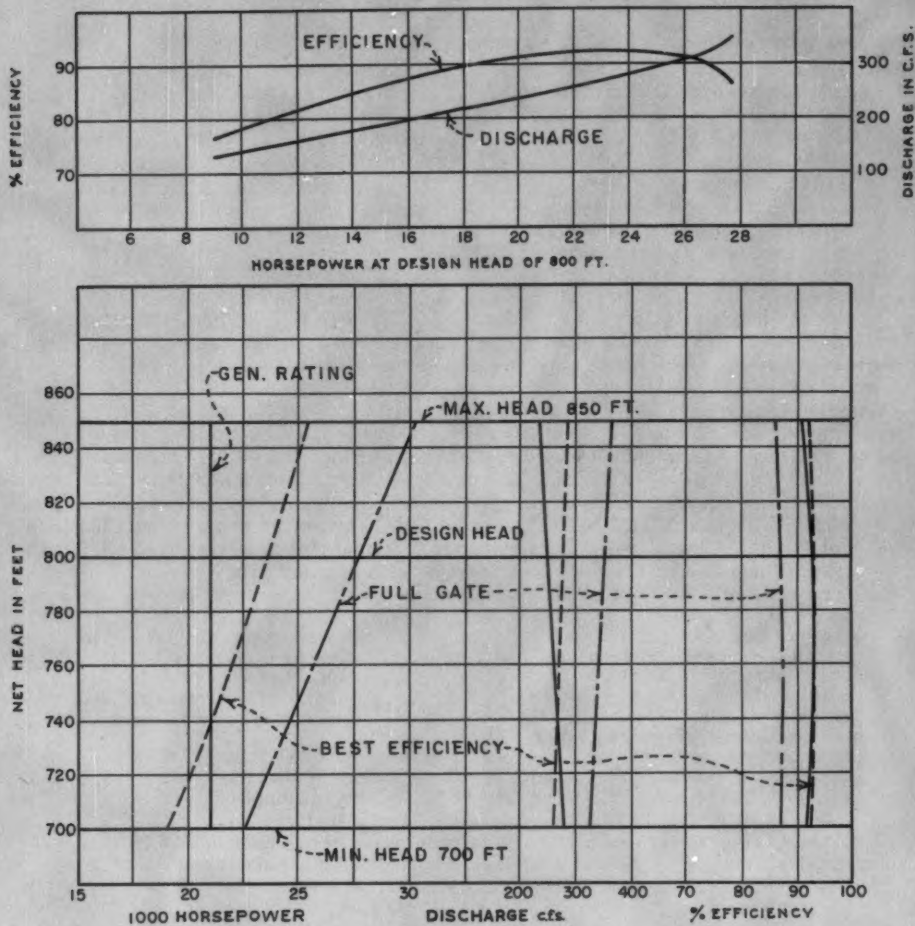
Intake Structure

The Eklutna Lake water is withdrawn at a level below operating storage, thus providing a free access for the water under the thick ice formed on the lake surface. (In the winter months the temperature may drop as low as minus 60 degrees.) Initial diversion from the lake is made through an inlet channel, 100 feet wide and about 500 feet long, excavated at the lake bottom at elevation 800 (about 60 feet below the lake surface elevation).

From the intake channel water enters a 133-foot 8-inch-long intake structure built in precast concrete sections for placing and joining together under water. The intake structure has an initial trashrack structure 112 feet long, built of rectangular precast concrete sections of varying depths. Each section is 26 feet 8 inches wide and about 37 feet long. Timber trashracks built of 2- by 8-inch planks spaced 2 inches apart are supported along the sections' upper surfaces. No ice or debris loading was assumed in the design of the trashracks and supporting walls; the trashracks were designed for erection dead loads only. The supporting walls were designed to withstand lateral earth pressure to the full height of the walls based on pressure for earth in water. No unbalanced water pressure was assumed. To assure proper inflow for the design capacity of 640 cfs when ice is on the lake, operating instructions require that the water should not be drawn down below elevation 814.

Immediately downstream from the trashrack structure is a precast concrete transition section, 14 feet 2 inches long. The inside of this transition section tapers from a rectangular reinforced concrete section 9 feet by 22 feet 8 inches to a circular section 9 feet in diameter.

At the end of the square-to-round transition section there is a bulkhead



PREDICTED CHARACTERISTIC CURVES

EKLUTNA POWER PLANT EKLUTNA PROJECT.

UNITS 1 AND 2

TURBINE RATING IN H.P. 25,000 RATED HEAD 800 FT. SPEED 600 R.P.M.

Generator rating in kv-a 16,667 Power factor 90 percent.

Turbine mfgr. Newport News Ship Bldg. & D.D. Co. Type Francis

Cost per unit f.o.b. factory $ 167,500 Weight 135,000 lbs.

Cost per h.p. $ 6.70 Weight per h.p. 5.4 lbs.

Type of scroll case SPIRAL-CAST STEEL - 2 Piece

Type of draft tube Elbow

Weight of runner 14,000 lbs.

Weight of turbine parts including hydraulic thrust to be carried by generator thrust bearing 99,500 lbs.

Governor capacity in foot-lbs. 42,000 Pipe size 3 inches.

Gov. mfgr. WOODWARD GOVERNOR CO. Time element 3 seconds.

Cost per unit f.o.b. factory $ 29,343 Weight 19,000 lbs.

Generator mfgr. Pacific Oerlikon

Generator WR^2 450,000 lbs. at one foot radius.

Turbine WR^2 18,500 lbs. at one foot radius.

Regulating constant of unit ($R.P.M.^2 \times WR^2 \div H.P.$) 6,750,000

n_s of runner 21.6 at 800 foot head when delivering 23,500 h.p. (Best eff.).

N_s of runner 23.5 at 800 foot head when delivering 27,700 h.p. (Full gate).

H.P. at 800 ft. (Design head) 27,700 ; at 100.0 percent of design head; 355 c.f.s.

H.P. at 850 ft. (Max. head) 30,300 ; at 106.3 percent of design head; 365 c.f.s.

H.P. at 700 ft. (Min. head) 22,500 ; at 87.5 percent of design head; 325 c.f.s.

H.P. at 800 ft. (Rated head) 25,000 ; at 100.0 percent of design head; 297 c.f.s.

H.P. at best efficiency equals 84.8 percent of h.p. at full gate.

Runaway speed at 830 ft. hd. 980 r.p.m. equals 163.3 percent of normal speed.

Dimensions of turbine:

Unit spacing 24 ft.

Max. dia. of runner 5.17 ft.

Dia. of gate circle 6.165 ft.

Height of distributor case 0.567 ft.

Dia. of scroll case inlet 3.75 ft.

Dia. of shaft 15 inches.

Dia. of cover plate 7.82 ft.

Number of wicket gates 18.

Number of stay vanes 18.

Dia. at top of draft tube 3.53 Ft. = D_3.

Outside radii of stay vanes 4.33 to 4.20 ft.

Distance from center line of distributor to top of draft tube 1.50 ft.

Depth of draft tube 11.00 ft. equals 312 percent of dia. D_3.

Length of draft tube 17.08 ft. equals 484 percent of dia. D_3.

Width of draft tube 12.00 ft. equals 340 percent of dia. D_3.

Distance from center line of turbine to center line of scroll case inlet 6.08 ft.

Distance from center line of distributor to minimum tailwater elevation (One unit operating at full load) 4.0 ft.

Figure 4(b)

Hydraulic turbine data sheet, Eklutna Powerplant

section 7 feet 6 inches long. Slots are provided in the bulkhead section for either stop planks or a fabricated bulkhead to be used in the event of an emergency or for inspection purposes. The slots extend above the trashrack section and are protected by a removable cover.

From the end of the bulkhead section the water is conveyed 225 feet through a 9-foot inside diameter precast concrete pipe extending to the entrance of the Eklutna Tunnel. The pipe has a wall thickness of 12 inches. It was cast in 16-foot sections, each section weighing about 40 tons. The sections were lifted at the casting site at the shore of the lake by a gantry crane which traveled on a wood-pile launching trestle as shown in Figure 5. The pipe sections were then lowered into the water at the end of the trestle, picked up by a floating barge, and transported to the intake site. They were then lowered about 75 feet into their final positions and were joined by divers. The sections are joined by steel clamp assemblies held together by 1-1/4-inch diameter wrought iron bolts, 27 inches long. Sand and gravel and waste material from the tunnel excavation were placed around the pipe sections and over the tops to a depth of 3 feet minimum.

Eklutna Tunnel

The Eklutna Tunnel is a major feature of the project. It is a circular concrete lined pressure tunnel (Figure 6) having an inside diameter of 9 feet and a length of 23,550 feet. Hydraulic properties of the tunnel are as follows: area—63.62 square feet; velocity—10.06 feet per second; capacity—640 cubic feet per second. Slope of the tunnel is 0.00341; the difference in elevation between the inlet and the outlet gate at the surge tank is 80 feet.

The tunnel proper begins at the downstream end of the precast pipe section. Gates are provided at two places in the tunnel—in a 9-foot-diameter bulkhead gate shaft (Figure 7) located about 600 feet downstream from the high water shoreline of the Eklutna Lake, and in the surge tank at the end of the tunnel. A construction adit, which intersects the tunnel immediately upstream from the surge tank, is used as a means of permanent access to the tunnel.

The bulkhead gate shaft is in an open cut, 725 feet downstream along the tunnel profile from the beginning of the tunnel proper. The shaft, about 209 feet deep, permits access to the tunnel downstream from the gate when the gate is down and the tunnel is dewatered. Descent in the shaft for inspection and maintenance is made by a series of staggered ladders and safety guards and platforms. The shaft is a reinforced concrete structure and has 15-inch walls anchored at the tunnel by reinforced concrete rectangular transition sections having 3-foot thick walls and bottom slab.

The bulkhead gate, 7.08 feet wide by 9 feet high, is a flat structural steel member of welded construction and has one cast phosphor-bronze seat on each side. These vertical seats support the water load on the gate and bear against seats on frames embedded in the downstream face of a vertical concrete slot in the gate shaft extending 22 feet up from the tunnel floor. The vertical seats act as metal seals in conjunction with a cast phosphor-bronze seal which extends across the top of the gate. Lateral movement of the gate is controlled by means of a finished surface on the gate bearing against the vertical slot. The gate is opened and closed by a hydraulic hoist which is direct-connected to the gate stem. The control system, located on a platform near the top of the gate shaft, is completely hand operated.

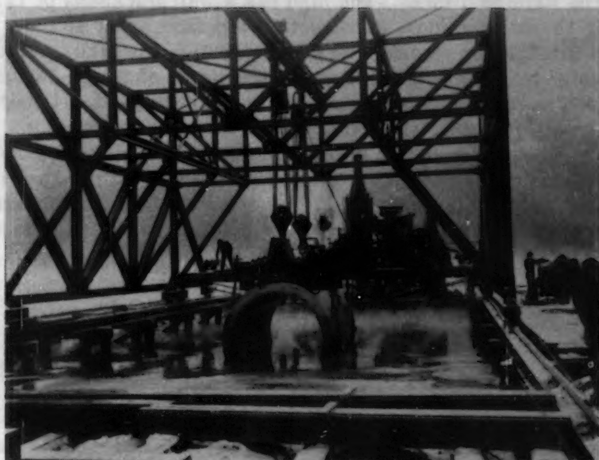


Figure 5

Shown here is the gantry crane lowering a section of the precast concrete pipe to the lake bottom.

Bureau of Reclamation photograph
No. P-783-908-1183

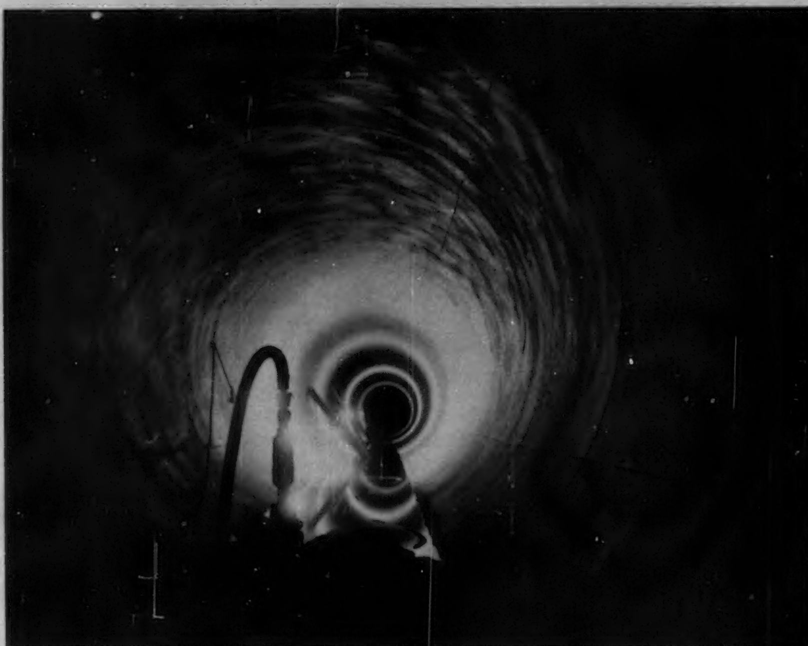


Figure 6

A view of the concrete-lined [illegible] Tunnel.

Bureau of Reclamation photograph

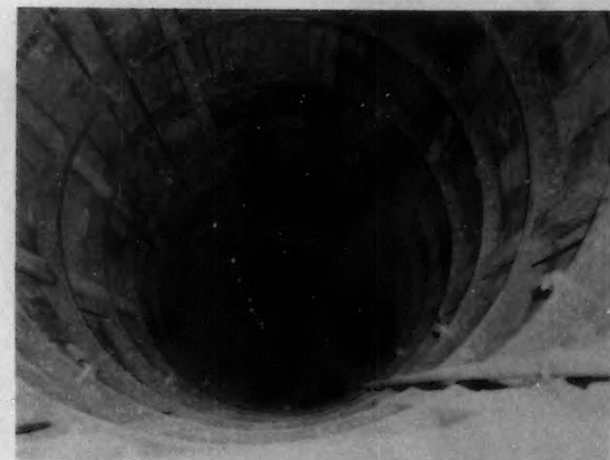


Figure 7

View down from the top of the gate shaft looking down 105 feet.

Bureau of Reclamation photograph
No. P-783-908-446

The tunnel was excavated through rock consisting principally of argillite and graywacke, ranging from fairly hard and massive to badly broken and crumbly. As described in the paper by W. R. Judd,[1] "The argillite resembles a compact clayey shale, highly fractured, with numerous slickensides, and is very soft in some drill cores. The graywacke is a hard, greenish, fine-grained sandstone that has undergone some metamorphism and is severely fractured."

The tunnel section extending from the intake section to the bulkhead gate shaft, a distance of about 711 feet, has a wall thickness of 12 inches, reinforced transversely by circular hoops of 1-inch steel and longitudinally by 1/2-inch bars at each face. For the remaining sections of the tunnel, the unsupported wall thickness is 9-1/2 inches. A total of 30,055 cubic yards of concrete and 2,353,402 pounds of reinforcing steel was placed in the tunnel lining.

Contraction joints are provided in the tunnel lining at every 25-foot interval in the 711-foot reach mentioned above. Permanent drains were placed in the tunnel to relieve excessive pore pressures in the surrounding rock. These drains discharge into the tunnel and are provided with flap valves to protect against water flowing out from the tunnel into the rock.

Surge Tank

The Eklutna Tunnel terminates in a surge tank installed directly over the tunnel 22,805 feet downstream from the bulkhead gate shaft. The surge tank is a reinforced concrete circular structure about 176 feet in height above the tunnel. The roof of the surge tank (Figure 8) is of reinforced concrete construction supported by structural steel members. The structure has an inside diameter of 30 feet and a wall 18 inches thick. Immediately downstream from the surge tank and built integrally with the tank wall is a 3-foot diameter access tube through which descent from the roof of the tank to the tunnel floor is made.

The tunnel section beneath the surge tank contains two 9-foot long square-to-round transitions spaced 4 feet 6 inches apart. The 4-foot 6-inch rectangular separation serves as the gate slot for the 7.08-foot by 9-foot fixed wheel gate which is used for emergency closure of the tunnel in the event of the damage of the penstock below or the turbines in the powerplant and for unwatering the penstock for inspection and maintenance. In an emergency the gate can be closed under full reservoir head flowing through the tunnel. The resulting water surge caused by the closure is absorbed in the surge tank.

The surge tank is of the restricted orifice type. Surge calculations for the tank were based on a flow of 690 cfs for maximum lake elevation and a flow of 640 cfs for minimum lake elevation. On the basis of elevation 724 at the orifice, the maximum upsurge elevation in the tank was calculated to be 902 when the flow is 690 cfs, and the minimum elevation of the hydraulic gradient in the tank with maximum downsurge was calculated to be elevation 739 when the flow is 640 cfs.

The fixed wheel gate is a flat structural steel member of welded construction having four flanged wheels on each side. The wheels support the water load on the gate and bear against stainless steel tracks that are mounted on

1. "Foundation Problems of the Eklutna Project" by W. R. Judd, A.M. ASCE, Separate No. 444, Proceedings ASCE, June 1954.

Figure 8

Downstream side of surge tank showing access shaft and platform. The contractor's temporary railway is shown in the foreground.

Bureau of Reclamation photograph

No. P-783-908-1729

bases embedded on the downstream face of the gate slot. The wheels are held against the tracks when the gate is closing by springs which bear against angles embedded in the upstream face of the slot. When the gate is in its normal open position, it is isolated in a concrete-walled chamber within the surge tank. This serves to maintain tunnel pressure differentials between the tunnel and the surge tank.

The fixed wheel gate is opened and closed by a hydraulic hoist which is direct-connected to the gate stem and is mounted in the top of the surge tank. The control system is mounted in a cabinet located in a house on the roof of the tank. (In the event of power failure, the gate can be operated manually.) An electrical interlock between the gate controls and the butterfly valves in the powerplant prevents the gate from being opened unless the valves are closed. The rate of opening of the gate is restricted so that the penstock will be filled before the flow under the gate has reached the maximum designed discharge rate. Thus, the surge produced in filling the penstock will be within the limits for which the system is designed.

Tunnel Adit

The tunnel adit is located at the outlet end of the tunnel near the surge tank. It is reached by an operating road extending up the mountain approximately 2 miles from the vicinity of the powerplant. The adit is essentially the same size as the main tunnel and is approximately 300 feet long, and provides one means of access to the tunnel for inspection and maintenance purposes. It also acts as a free-flow conduit in conveying drainage water from the tunnel when entrance into the tunnel is necessary, and also when it is desired to prevent the water from flowing down the penstock. Access from the adit to the tunnel is provided by a water-tight door.

Penstock

Extending from the surge tank at the end of the Eklutna Tunnel is the power penstock which conveys water to the powerplant turbines. The overall length of the penstock is about 1,088 feet, installed in 30-foot sections. The penstock is a variable-diameter, 91-inch, 83-inch, and 75-inch (outside diameter) welded and coupled steel pipe encased in concrete in a tunnel extending from the surge tank to the powerplant. Plate thickness of the penstock varies from 5/16-inch at the initial section to 1-1/2-inch at its terminal section. In profile, the penstock roughly parallels the mountainside, descending for approximately 864 feet at an angle of 53 degrees; it then levels off and continues through a horizontal section about 501 feet long. Erection of a penstock section is shown in Figure 9.

The penstock bifurcates into two 51-inch diameter 23-foot long branches at the powerplant which are connected to the spiral cases of the turbines. A 66-inch butterfly valve is installed in each penstock branch upstream from the turbines to provide means of unwatering the turbines for servicing or maintenance. These valves also serve as emergency shutoff valves in the event of damage to the turbines.

The penstock is vented at the gate structure in the surge tank and immediately downstream from each butterfly valve to allow air to escape when the penstock is filling, and to prevent negative pressures from occurring when the gate is closed in an emergency or when the tunnel is being drained. Access to the penstock interior is obtained through the vent at the surge tank,

Figure 9

Photographed here is one of the two hoists suspended on opposite sides of the excavation which were used in positioning the steel penstock on the incline.

Bureau of Reclamation photograph
No. P-783-908-1614

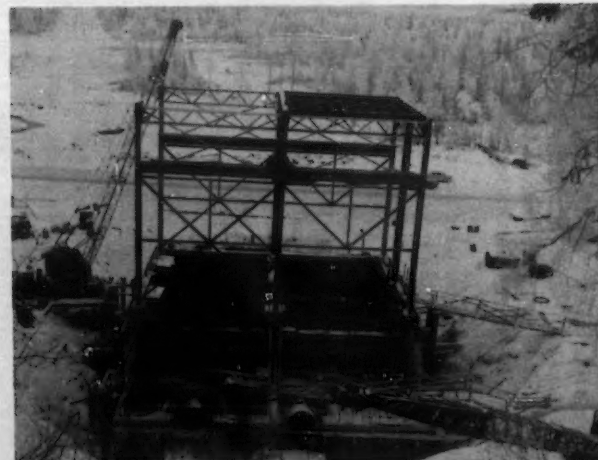


Figure 10

View of steel erection for powerhouse superstructure.

Bureau of Reclamation photograph
No. P-783-908-1532

through manholes in the powerplant, and through the tunnel adit.

The penstock is filled by opening the fixed-wheel gate in the surge tank at the restricted rate of opening, thus allowing the penstock to fill slowly without causing pressure surges in the penstock. Air displaced by the water entering the penstock is released through the vent at the surge tank.

Powerplant Structure

The Eklutna Powerplant is a reinforced concrete structure housing the generators and turbines and related electrical, mechanical, and hydraulic equipment. Sections through the structure are shown in Figures 3(a) and 3(b). It is 71 feet wide and 73 feet 8 inches long. Overall height of the building from foundation to the top of the roof is 86 feet 7 inches. The 35-foot deep substructure is of reinforced concrete construction; the walls are 3 feet thick and the subfloor slab is 5 feet thick.

Adjacent to the west wall of the powerplant is a machine shop, 78 feet long, 27 feet wide, and 20 feet high. Built integrally with the powerplant structure at its south wall are a control room and electrical equipment room, each 35 feet 6 inches long, 25 feet wide, and 17 feet high. South of the machine shop and adjacent to the west wall of the powerplant are three open transformer bays separated by 12-inch reinforced concrete walls.

The foundations for the powerplant structure, transformer bays, and penstock anchor block are supported by 14-inch, 73-pound steel H-bearing piles. A total of 18,597 lineal feet of piling was driven, consisting of 289 vertical and 110 batter piles (on a 1:4 batter). All piles were driven to bedrock. Permanent bench marks were established in the concrete floor at elevation 15.00 to check settlement of the structure. Levels were run periodically during construction and also since the plant has been in operation but no settlement has occurred to date.

The superstructure of the powerplant has structural steel framing and reinforced concrete walls 12 inches thick insulated by a 2-inch layer of insulating board. Erection of the structural steel is shown in Figure 10. The height of the superstructure is 51 feet. Principal structural members of the superstructure are three rigid frame steel bents built up of 36-inch, wide-flange steel columns to support the plant's crane runway for a 40-ton capacity crane and a 4-foot 4-inch horizontal plate girder supporting the roof purlins and the 6-inch concrete roof slab. The bents are braced by 10 WF 21 structural steel members placed diagonally between columns. The machine shop is also framed by structural steel members and has 10-inch reinforced concrete walls insulated with a 2-inch layer of insulating board. A total of 286,811 pounds of structural steel was used in the building frames.

An electric cab-operated overhead traveling-type crane, having a span of 40 feet 10 inches, operates on a runway which extends the length of the powerplant. The crane, used for installing and maintaining the generators, turbines, and other equipment, has a 40-ton capacity main hoist powered by a 30-horsepower motor.

A jib crane having a 2-ton capacity twin lift chain hoist is attached to the powerplant wall between the draft tube bulkhead gate slots for raising, lowering, pivoting, and transferring the draft tube bulkhead gates from one slot to the other. A hand-racked, underslung traveling-type 3-ton capacity crane operates on a runway which extends the length of the machine shop. The crane is used for handling materials and equipment in the shop.

Turbines

The powerplant has two vertical shaft, Francis-type turbines having cast steel spiral cases and elbow-type draft tubes. Each turbine has a rated capacity of 25,000 horsepower at full gate opening when it is operating at 600 rpm under a net head of 800 feet. At this head and an output of 23,000 horsepower the warranted efficiency of the turbine is 90 percent. At rated head and full gate output the predicted discharge from one turbine is 330 cfs.

The turbines are required to operate satisfactorily at effective heads between 700 feet and 850 feet. The controlling design water surface elevations are as follows:

Headwater	Elevations (Forebay)
Maximum	867.5
Average	852.0
Minimum	814.0
Tailwater	**Elevations**
Maximum	35.0
Minimum	20.0

Turbine operating characteristics and design information are shown in Figures 4(a) and 4(b), hydraulic turbine data sheet.

Generators

The two generators in the powerplant (Figure 11) are each rated 16,667 kva, 90 percent power factor, 6,900-volts, 3-phase, and 60-cycle at 600 rpm. Each generator is a vertical shaft type synchronous machine, designed for clockwise rotation when looking down on the unit. A thrust bearing and an upper guide bearing are provided above the rotor; a power guide bearing is provided below the rotor. A common oil reservoir is provided for the thrust and upper guide bearing, and there is a separate oil reservoir for the lower guide bearing. As the thrust bearing is supported by a ring mounted on a bracket consisting of a bridge of two steel beams supported from the stator frame, the above load is transmitted successively through this bracket to the stator frame, then to the sole plates and on into the foundation. The upper bearing bracket also supports the main exciter, which is directly coupled to the generator shaft. The lower bracket supports the lower guide bearing and the combination air brakes and hydraulic jacks.

The stator bore is 7 feet 10-31/64 inches in diameter. This allows clearance for installation or replacement of any part of the generator or turbine located below the stator including the lower guide bearing bracket of the generator.

Each generator is excited by a direct-connected vertical-shaft, shunt wound type, d-c generator mounted on top of the generator. Both generators are equipped with an enclosed cooling system, complete with air ducts, five separate water-cooled heat exchangers located around the periphery of the generator stators and metal housings. Blades attached to the rotor act as blowers to circulate air.

The generators are equipped with air-operated brakes mounted on the lower bearing bracket. The brakes are of sufficient capacity to bring the

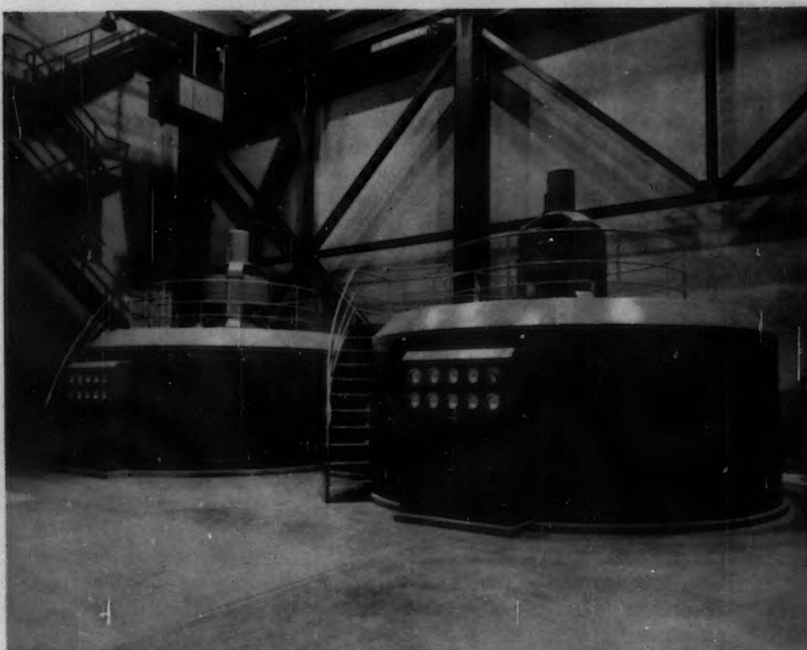


Figure 11

Interior view of the Eklutna Powerhouse showing the two 15,000-kw generators.

Bureau of Reclamation photograph No. P-783-908-1761

rotating parts of the generator and turbine to a stop from 1/2 normal operating speed within 7-1/2 minutes after the brakes are applied. The brakes are also designed for use as hydraulic jacks to lift the generator and turbine rotating parts during generator disassembly.

Switchyard

The switching equipment for the powerplant is located at three different elevations. The switchyard equipment itself, consisting of the power circuit breakers, disconnecting switches, and main busses, is on the roof of the powerplant at elevation 92.50 (Figure 12). The main power transformers that "step up" the generator low voltage are located in the transformer bay adjacent to and southwest of the powerplant structure at elevation 41.25. From the high-voltage bushings of these main power transformers, a bus-work is used to the main switching equipment located on the roof; this is accomplished by means of an intermediate or "tie bus" which is located on the roof of the control room at elevation 58.54.

The 115-kv bus structure on the powerplant roof consists of two bays to supply the 115-kv lines to the cities of Palmer and Anchorage. In addition, there is a 12.47-kv line which supplies power to the Government camp from a small transformer energized from the low-voltage generator leads. This transformer is in the transformer bay adjacent to and south of the powerplant structure at elevation 41.25.

The lines to Palmer and Anchorage are equipped with 115-kv circuit breakers of the air-blast type. Disconnecting switches are placed on both sides of the breakers, to permit removal of the breakers from service for needed maintenance or repairs.

The 115-kv and 12.47-kv steel switchyard structures were designed to withstand the loads imposed by line and transformer circuit conductors and ground wires and tension busses, the loads due to wind and dead load, and the loads imposed by the installation of electrical equipment.

Draft Tubes

Each turbine has an elbow-type draft tube, the elbow portion of which has a welded plate-steel liner. A 6-inch drain connection is provided at the low point of the liner for unwatering the draft tube.

There is one draft tube bulkhead gate 12 feet wide by 4 feet high provided for use in sealing off the draft tube openings when a turbine is shut down and unwatering of the draft tube is required. The gate is lowered, raised, and transferred from one opening to the other by the 2-ton capacity, hand-operated, twin-lift hoist suspended from the jib crane attached to the wall of the powerplant superstructure. In the event high-pressure penstock water is unintentionally admitted to the turbine draft tube while the gate is in place, the gate will be blown off its seat because of the relatively few bolts which secure the shear angle to the gate. This is a safety feature intended to protect the gate or prevent a possible failure of either the turbine-head cover or draft tube man door and subsequent flooding of the powerplant.

Tailrace

Water discharged from the draft tubes of the turbines in the powerplant enters a 209-foot long pressure tailrace conduit through which the water is

Figure 12
Photographed here is the completed Eklutna Powerhouse

Bureau of Reclamation photograph
No. P-783-908-1755

Figure 13
Tailrace conduit, showing partially completed compacted backfill.

Bureau of Reclamation photograph
No. P-783-908-1338

conducted under the Glenn Highway to an open tailrace channel which discharges into the Knik River. A surge chamber is provided at the entrance to the pressure conduit to store water resulting from increases in load demand until the water in the conduit can be accelerated. This serves to prevent excessive pressure build-up in the draft tubes.

The tailrace conduit (Figure 13) is made up of rectangular reinforced concrete transition sections having varying widths and depths. The terminal section of the conduit is 50 feet long and flares outward in the downstream direction from a width of 14 feet 6 inches to a width of 46 feet 6 inches. This terminal section is also of varying depth and has five openings separated by 10-inch walls through which the water passes into the tailrace channel. Stop log slots are provided at the outlet of the conduit. The stop logs are available for use when it is necessary to dewater the conduit or to unwater both draft tubes at the same time.

The banks of the open tailrace channel are built on a 2 to 1 slope and are lined with riprap at its junction with the tailrace conduit. The channel has a top width of about 75 feet, a bottom width of 25 feet, and a height of about 12 feet 6 inches.

Although Knik Arm is a tidal estuary, the tidal conditions in the Arm slightly below the point of confluence of the tailrace channel with the Knik River were considered to have no effect upon operations of the water discharge system.

Transmission Lines

Table 1 gives data on the transmission lines of the Eklutna Project.

Table 1

TRANSMISSION LINES ON THE EKLUTNA PROJECT
OPERATED BY THE BUREAU OF RECLAMATION

Line Terminals	Voltage (kv)	Capacity (Megawatts)	Conductors	Supporting Structures	Circuit Miles	Placed in Service
Eklutna Powerplant - Anchorage Substation	115	(a) 30	ACSR 397.5 MCM	Wood H-frame	32	April 1953
Eklutna Powerplant - Palmer Substation	115	(a) 5	ACSR 397.5 MCM	Wood H-frame	9	October 1952
Little Eklutna Powerplant - Anchorage (b)	34.5	2.5	Cu #4	Wood pole	24.3	
Eklutna Powerplant - Government Camp	12.47	(a) 0.750	ACSR #4/0	Wood pole	1	June 1952

(a) Transformer ratings

(b) Line acquired from City of Anchorage presently used by City to supply Bureau of Reclamation requirements at the old Eklutna plant. The Bureau also wheels power for the Civil Aeronautics Administration station for the City of Anchorage over this line.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

DESCRIPTION OF REPAIRS TO SPILLWAY PIERS OF KEOKUK DAM

Ralph L. Shelton,[1] and Frank L. Burgrabbe[2]
(Proc. Paper 1133)

SYNOPSIS

This paper describes the rehabilitation of the spillway piers of this dam after more than 40 years of service. Deteriorated concrete was removed and replaced with temperature-reinforced concrete anchored to sound concrete of piers.

INTRODUCTION

After more than 40 years of service the spillway piers of Keokuk Dam showed evidence of severe erosion and deterioration. A thorough field investigation in conjunction with extensive field and laboratory testing indicated the need for repairs.

Erosion was most pronounced at the intersection of the spillway ogee with the piers. Concrete cores taken in the field indicated deterioration of the concrete on the wetted surface of the pier, directly above the ogee intersection.

Figure 1 shows one of the piers before repairs were made. The dark stained area above the spillway indicates the deteriorated area.

The repairs consist of removing the deteriorated concrete down to sound concrete and replacing it with a durable concrete armor coat.

Fruin Colnon Contracting Company of St. Louis, Missouri are contractors for the repairs and although their construction plant was set up in late fall of 1954, they were unable to start repairs that year because of high tail water and cold weather.

In May of 1955, the Contractor resumed operations and on June 16, the first pier repair was poured. On November 10 repairs had been completed

Note: Discussion open until May 1, 1957. Paper 1133 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 6, December, 1956.

1. Structural Engr., Union Electric Co. of Missouri, St. Louis, Mo.
2. Union Electric Co. of Missouri, St. Louis, Mo.

on 75 of the 120 piers. On December 1 operations were discontinued until the spring of 1956.

Construction Plant

The compressed air plant is located at the east end of the dam in the Hamilton, Illinois Yards which is the base of all construction activities. It consists of 2-625CFM air compressors, each driven by a 125 H.P. motor. A 6" pipe line, connected to the compressor plant, extends approximately 4,400 feet across the full length of the spillway deck and provides for compressed air takeoff as repairs move across the spillway.

Hoisting from the spillway deck was done with the plant locomotive crane, driven by compressed air. Due to the presence of overhead power lines, the crane was "flat boomed" to avoid interference with them. Although the "flat boomed" capacity of the crane was only 4 1/2 tons, it was sufficient except for several times when excavated concrete jammed against the sheet pile guide frames and more capacity was required for lifting. To overcome this, outriggers were used which increased crane capacity to 15 tons.

For moving concrete across the dam, a ready mix truck is driven up a movable ramp onto a railroad car, which is coupled to a "Trackmobile." Figure 2 shows the ready mix truck up the ramp onto the railroad car. The Trackmobile was also used for moving the locomotive crane on and off the spillway deck.

As all the repairs were downstream, dewatering was accomplished by driving a single wall, steel sheet pile cofferdam approximately 5 feet downstream from the toe of the spillway bucket. The sheeting was driven against a structural steel guide frame which was anchored to the spillway ogee. Between 8 and 10 spillways are dewatered at one time.

Two 10" and one 4" electric driven pumps and two 6" gasoline driven pumps are used for dewatering. In addition, several 2" air driven siphons are used to dewater local areas in the cofferdam. Figure 3 shows a view of the cofferdam.

Pier repairs started at the east end of the spillway and progressed westward toward the power house. As repairs move westward, sheet piling is pulled and redriven to keep in step for dewatering. An abandoned 33 kv overhead line on the downstream side of the dam was energized with 2,300 volt 3 phase 60 cycle power. It serves as a continuous bus across the spillway to which two portable 2,300/440 volt step down transformers are connected. These transformers serve the dewatering pumps and this arrangement provides for intermediate tap off connections as the work progresses across the dam. Figure 4 shows the overhead bus. The transformer can be faintly seen in the background.

Three steel barges, one 18' x 58', one 12' x 47' and one 12' x 40' are moored directly downstream from the cofferdam. The two largest barges are used for storing sheet piling and metal forms while the smaller one provides a convenient portable platform for the cement finishers use.

Concrete Cutting and Placing

Access to the sides of the piers for concrete cutting is provided by wooden stairs placed on the spillway. Overhead trolley beams support chain hoists

which are used for final placing of metal forms after initial positioning by the locomotive crane. 1 1/2" pipe sleeves which extend thru the full 6'-0" width of the piers and which were used as form spreaders on the original dam construction, serve as a convenient location for placing 1" rods which support the balancers for the cutting hammers. Figure 5 shows cutting operations.

On the sides of the piers a minimum cut of 6" is specified. On the 75 piers repaired to date, the 6" cut has been ample to reach sound concrete at the top of the deteriorated area. The maximum cut required thus far to reach sound concrete has been 12".

The first 4" to 6" inches of old concrete removed came off like mud but when sound concrete was reached, it was readily determined by a decided increase in resistance to cutting and by a definite change in color. Numerous mud balls varying from 2" to 6" in diameter were encountered during concrete removals on the first ten piers. All concrete permanently below the water line was extremely hard and difficult to cut. The first 10 spillways dewatered had a layer of mud about 4'-0" thick on top of the spillway aprons. When the mud had been removed and concrete cutting was started, this mud covered concrete was almost as hard as steel and had a very pronounced ring when struck with a hammer. Its color was dark blue, almost black.

One interesting discovery was the finding of a hole through the full width of one of the piers, large enough for a man to crawl through. Figure 6 shows this hole. Needless to say, it was dry packed before concrete cutting began on this pier.

Reinforcing dowels 3/4" in diameter and 30" on centers in both directions were grouted 17" into the existing concrete and 16" into the new armor coat. Temperature reinforcing in the new concrete consisted of 1/2" diameter bars 12" on centers in both directions. Figure 7 shows reinforcing in place on one of the piers.

After concrete cutting was completed, small loose particles were removed by a compressed air and water spray. The average concrete removed on the first 75 piers was slightly in excess of 20 yards each.

On the first 8 piers as a safety measure, 2 1/4" diameter steel pins were grouted into the spillway at the ends of the downstream face of the crest gates and the downstream face of the pier was poured before cutting started on the sides. By the time the first 8 piers were completed, sufficient confidence in the stability of the structure had developed and the pins were discarded and cutting of both sides and the downstream face were done simultaneously thereafter.

Concrete is delivered to the top of the dam as previously described, by a railroad car carrying the ready mix truck. A long string of tremie cans extends down to pour pockets in the steel forms. Six pour pockets are provided in the side forms and one at the top of the downstream form.

A 10 man crew was used for placing the concrete. Two at the top of the dam, handling the concrete from the truck to the tremies and 8 men below handling tremies at the pour pockets and vibrating forms. An external form vibrator is fastened to the form and internal vibrators are used at pour pockets; one at the pocket where the pour is being made and one at the pocket below. In addition an air chipping hammer, with a flat horizontal bar welded to the bit is used below both of the internal vibrators to provide further vibration.

Some difficulty was experienced with honeycomb on the first piers poured when a 3" slump concrete was used. After considerable experimenting, the

conclusion was reached that a 5" to 6" slump was required to avoid excess honeycombing. With the larger slump, honeycombing was greatly reduced.

Curing commenced immediately after forms were stripped and continued for 7 days. A 1/2" galvanized pipe, capped at the ends and perforated is suspended above the finished pour. It is connected to siphon which extends up to the upstream pool.

A 2" wide by 6" deep slot is provided at the top of the new concrete to provide for a dry pack joint between old and new concrete. It was formed out at first with a celotex filler but this proved troublesome in cleaning it off the new concrete and the Contractor later switched to a 2 piece plywood filler. Figure 8 shows the concrete armor coat poured with the dry pack slot at the top.

The dry pack material consisted of a 1-3 cement, sand mortar with just enough water added to make it stick together when squeezed in the hand. It was rammed into place with an air hammer fitted with a flat horizontal bar welded to the fill bit. A 1/2" finish coat of 1-3 mortar was applied after the dry pack joint was placed. Figure 9 shows a completed pier with the dry pack joint in place.

Progress was slow at first but after experience had been gained in handling the metal forms; coordinating moving of cofferdams and dewatering pumps, the Contractor was able to repair a pier per day. Of the 75 piers repaired, 47 were consecutively poured, "one a day."

Concrete Mix and Strength

Fly ash concrete is used for the armor patch and consists of 4.8 sacks of cement plus 113 lbs. of fly ash per yard of concrete. In addition, an air entraining agent was added during mixing. Sand and gravel conforming to ASTM Specification C-33 were used as fine and coarse aggregate.

A compressive strength of 3,500 psi at 28 days was required. Concrete 28 day strengths averaged 4,400 psi.

CONCLUSIONS

The removal of the deteriorated concrete from the piers has been most satisfactory and verifies predicted depths of deterioration. It was demonstrated that sound concrete existed a reasonable depth below the deteriorated pier face and that it was practical to apply an armor coat to it.

The bonding of the fly ash to the existing stone concrete is excellent and test results of the fly ash concrete exceeded anticipated strengths.

It is felt that the repairs will endure for many years and immeasurably prolong the life of the dam.

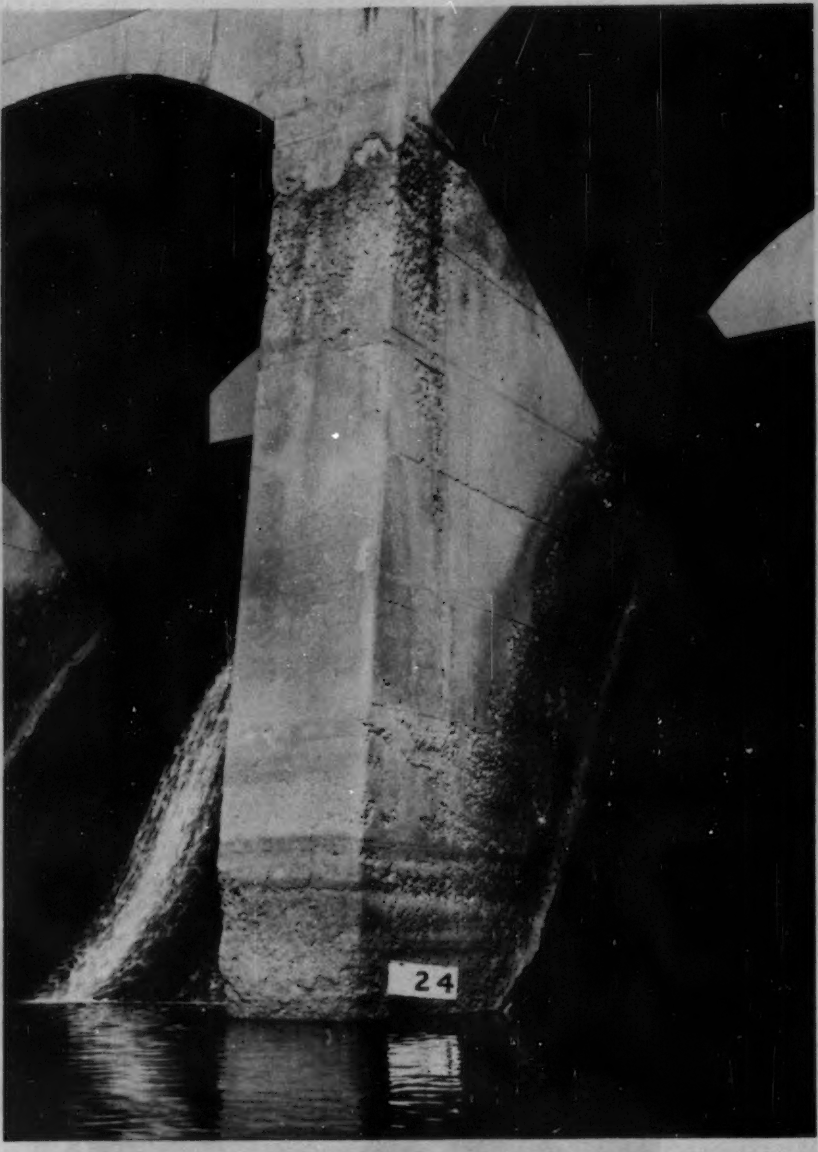


FIGURE I SHOWING DETERIORATION AND EROSION OF PIER.

FIGURE 2 SHOWING READY MIX TRUCK UP THE RAMP TO THE RAILROAD CAR

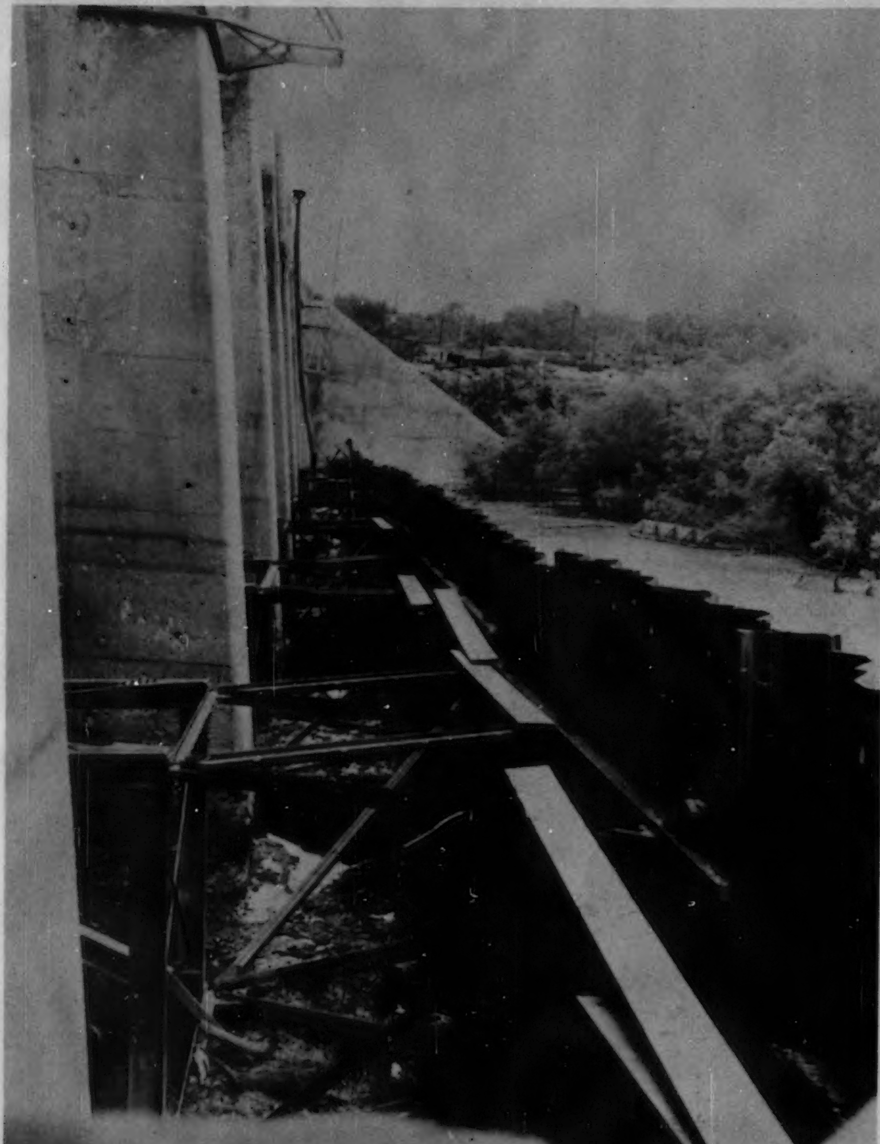


FIGURE 3 SHOWING VIEW OF COFFERDAM

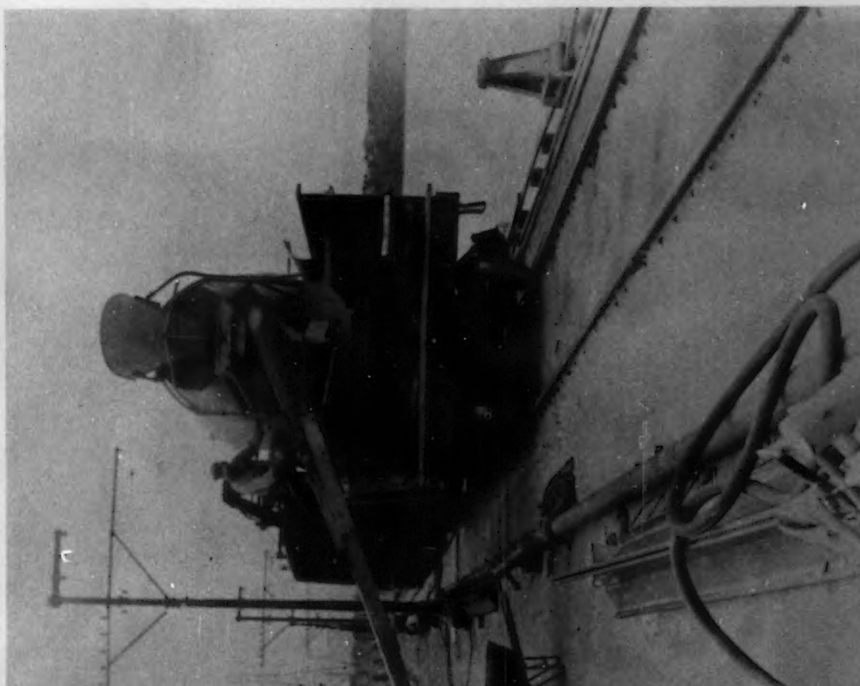


FIGURE 4. SHOWING OVERHEAD POWER SUPPLY LINE.

FIGURE 5 SHOWING CONCRETE REMOVED FROM SIDE OF PIER NO. 7.

FIGURE 6 SHOWING HOLE THROUGH ENTIRE WIDTH OF PIER NO. 25.

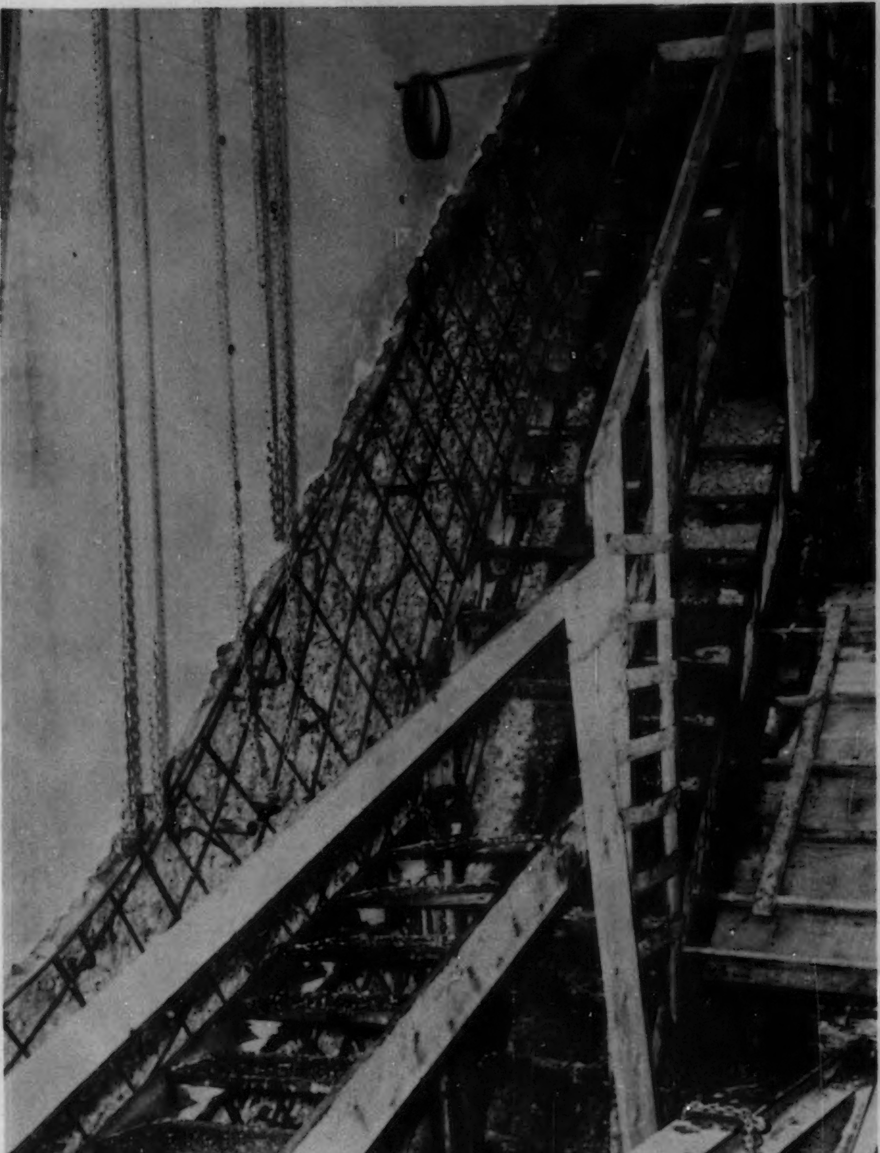


FIGURE 7 SHOWING REINFORCING IN PLACE ON SIDE OF PIER NO. 7

FIGURE 8 SHOWING CONCRETE REPAIR IN PLACE WITH DRY PACK JOINT EXPOSED.

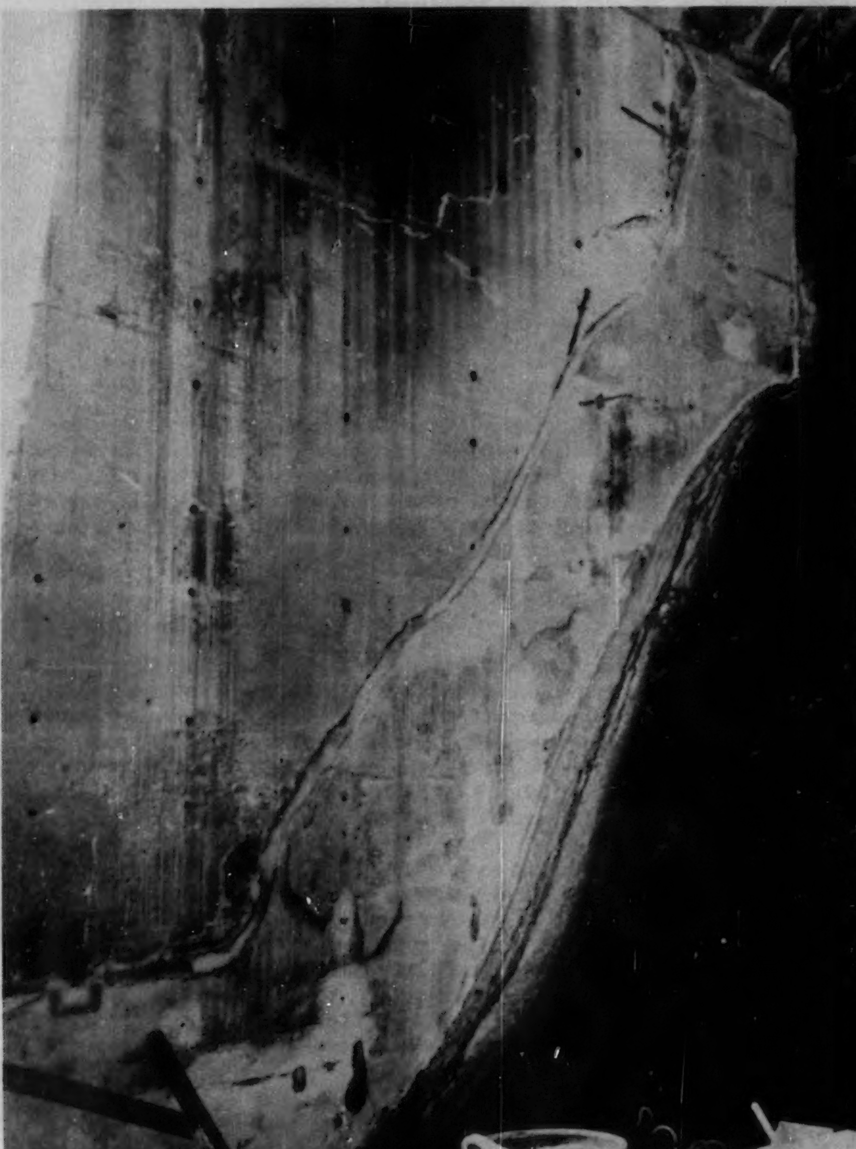


FIGURE 9 SHOWING COMPLETED REPAIR WITH DRY PACK JOINT IN PLACE

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OBSERVED BEHAVIOR OF SEVERAL ITALIAN ARCH DAMS

Professor Dino Tonini,[1] M. ASCE
(Proc. Paper 1134)

ABSTRACT

A summary of the results obtained from the analysis of deflection, strain, temperature and other measurements on nine Italian Arch Dams is presented. The important way in which temperature changes influence arch dam behavior is shown. The results of testing for foundation modulus by seismic methods before, during and after construction of arch dams are described.

1) - Premises

1. 1) The Società Adriatica di Elettricita among a total of 25 dams has at present in service or under construction the following large arch dams (between brackets, year when the work was completed), descriptions of which may be found in various papers already published or in course of publication: 1) Comelico (1931), 2) Maina di Sauris (1947), 3) Pieve di Cadore, archgravity dam (1949), 4) Valle di Cadore (1950), 5) Val Gallina (1951), 6) Barcis (1953), 7) Ambiesta (in course of completion), 8) Pontesei (in course of completion), 9) Vajont, arch dam (under construction).

Each dam has been provided with a more or less extensive system of control which normally includes the following measurements: a) temperature and humidity in the interior of the structure; b) deflections of characteristic points of the structure and profiles of the basin upstream and downstream of the dam, by means of large range collimators, pendulums, inclinometers and high-precision triangulations and levelling surveys; c) strains and stresses in characteristic points of the structure by means of rosettes of strain and stress meters; d) various phenomena, such as the variations of uplift pressures, elastic properties of the rock, of the structure, etc.

Note: Discussion open until May 1, 1957. Paper 1134 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 82, No. PO 6, December, 1956.

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Close to each dam there is, as a rule, a meteorological station; near the Pieve di Cadore and Ambiesta dams a seismographic station has also been set up.

The measurements, as far as possible, are remote recorded and concentrated in a single station situated in the central control room of the intakes of the dam. This solution requires a greater length of cable than would be necessary with more measurement stations, but has been preferred on the grounds that because of the local conditions in which the dam's staff live, convenience of measurements is the basis for their success. The system of control is studied jointly by the departments charged with the planning and construction of the dam and those in charge of its operation.

In general we try to study a single phenomenon by means of different methods of measurement so as to achieve, from the possible differences that cannot be imputed to the precision of the measurement, information on the complementary phenomena which accompany what at the first approximation was considered to be the main phenomenon.

The observations are begun during the construction and continued without interruption after the dam is in service, more or less frequently, according to the sensivity of reaction of the structure to the various actions to which it is subjected.

A special office superintends the various measurements, collects them, and interpretes them in order to make the transition from the numerical figures of the observations to figures which may have a physical significance in relation to corresponding phenomena.

Actually, not all measurements can be easily and immediately interpreted, because, as it is known, they are generally the result of many factors that are not readily distinguishable. However, from the numerous series of observations available, some of the most typical have been selected concerning the functioning of dams as arch structures. The aim of this paper is, therefore, to provide some information on this subject.

1. 2) In general, to get observations of some significance, averages have been taken usually at intervals from three to thirty days. In fact, the correlation between instantaneous cause and effect cannot, in most cases be easily determined because of the multiplicity of causes which not always produce the same effects and because of the way the effects themselves occur. Besides, the search for the most probable value of the several elements in question would involve many readings for every measurement, to determine it.

In other words, given the type of phenomena under examination, the features of the instruments and the possibilities of the observers, it has been held that, in the first stage of research, an average of, for example, ten figures read successively on ten different days and so corresponding to the average conditions of the structure in that period of time, is more interesting than an average of ten values corresponding to a particular moment. The latter is an average which can give us a more accurate measurement of the state of the phenomena in that single moment, but does not allow a more general interpretation.

For geodetic survey, on the other hand, the usual procedure of compensation is adopted but duly reduced, since, excluding the first measurement, we are not concerned with the absolute figures of the various sizes, but only with the differential ones. We must note, however, that geodetic surveys also

refer to an average condition of the structure corresponding to the interval of time included between the beginning and the end of the measurements. This interval, for fairly extensive triangulation nets is always of some days.

Lastly, we remind you that for some investigations mobile averages taken over a period of one year have been adopted. In fact some of the most important actions affecting the structures (temperature of the air, and the water, variations of load, etc.), show seasonal periodical variations within the year; the average values of these may be considered in many cases to be practically constant, taking into account also the attenuation in reflected phenomena. Now, if this constancy is not found in the annual mobile averages of the supposed effects (temperatures, deflections, strains, etc., of the structure) the fact can be deduced that other non-periodical phenomena have been added to the former (development of setting heat, creep of the concrete and of the rocks, inbibition of the rocks themselves, etc.). We thus have a very simple method of evaluating, in many cases, the entity of these non-periodical phenomena and in any case their progress.

2) - Measurements of Temperature

2. 1) The measurements of temperature of concrete structures are known to be among the simplest and quickest. There is also a good agreement between experimental results and those deduced from the application of Fourier's theory, when the range of external temperatures (air and water), the heat and specific weight of the concrete together with the coefficient of thermal diffusion are known.

In a given structure the number of instruments may therefore be reduced to a few for reference and control, concentrated mainly near the faces and the middle where, because of the possible interference of thermal waves coming from upstream and downstream, experimental figures less in agreement with the theoretical ones are obtained. An example of the correspondence between theoretical and experimental results is illustrated for the Lumiei dam in Fig. 1.

2. 2) The method of mobile averages has been adopted in order to determine the end of the period of dissipation of the setting heat of the concrete, being supposed, as mentioned before, that the average annual range for the temperature of air and water do not vary greatly from one year to the next.

When these mobile averages of temperatures in the interior of a dam have an asymptotic trend, it is right in assuming that the setting heat is practically exhausted: in Fig. 2 one example of this diagram is shown which illustrates how for the Pieve di Cadore dam the asymptotic trend is reached at the middle of a section of 19,3 m thickness after about 40 months.

2. 3) To give an idea of the annual ranges of temperature in the interior of the concrete dam, in the diagram of Fig. 3, for some of the structures in question, the figures checked at the middle of various sections, are recorded. These figures are not necessarily the lowest for section under examination since, at the middle there are often interferences of thermal waves from upstream and downstream.

The diagram clearly shows how for the Pieve di Cadore and Val Gallina dams, which have the same technological qualities (cement and aggregates), not very dissimilar graphs are obtained; the Lumiei dam on the other hand, differs in its behaviour having different technological and climatic features.

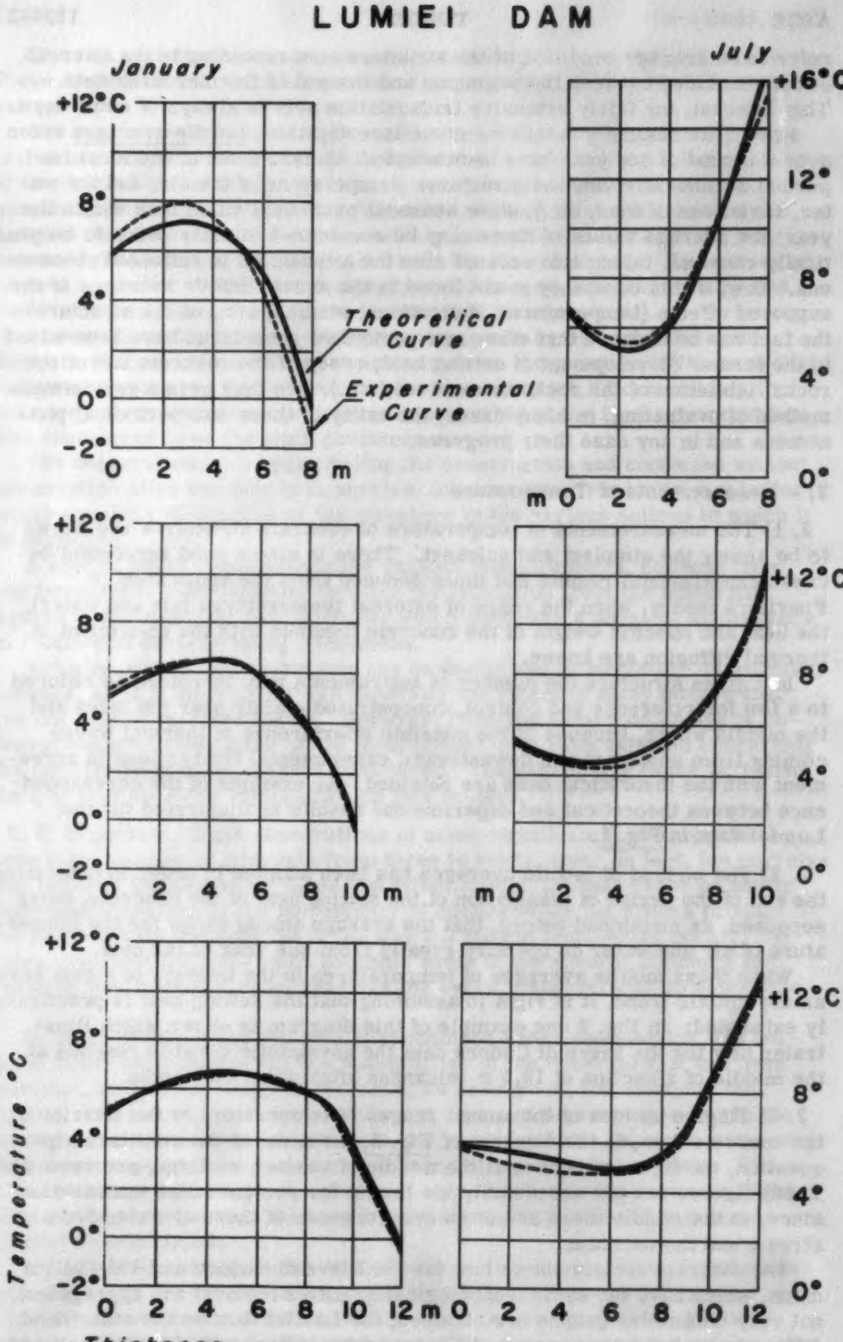

Fig. 1. Lumiei dam: distribution of temperatures, comparison between theoretical and experimental results with maximum and minimum external temperatures (July, January).

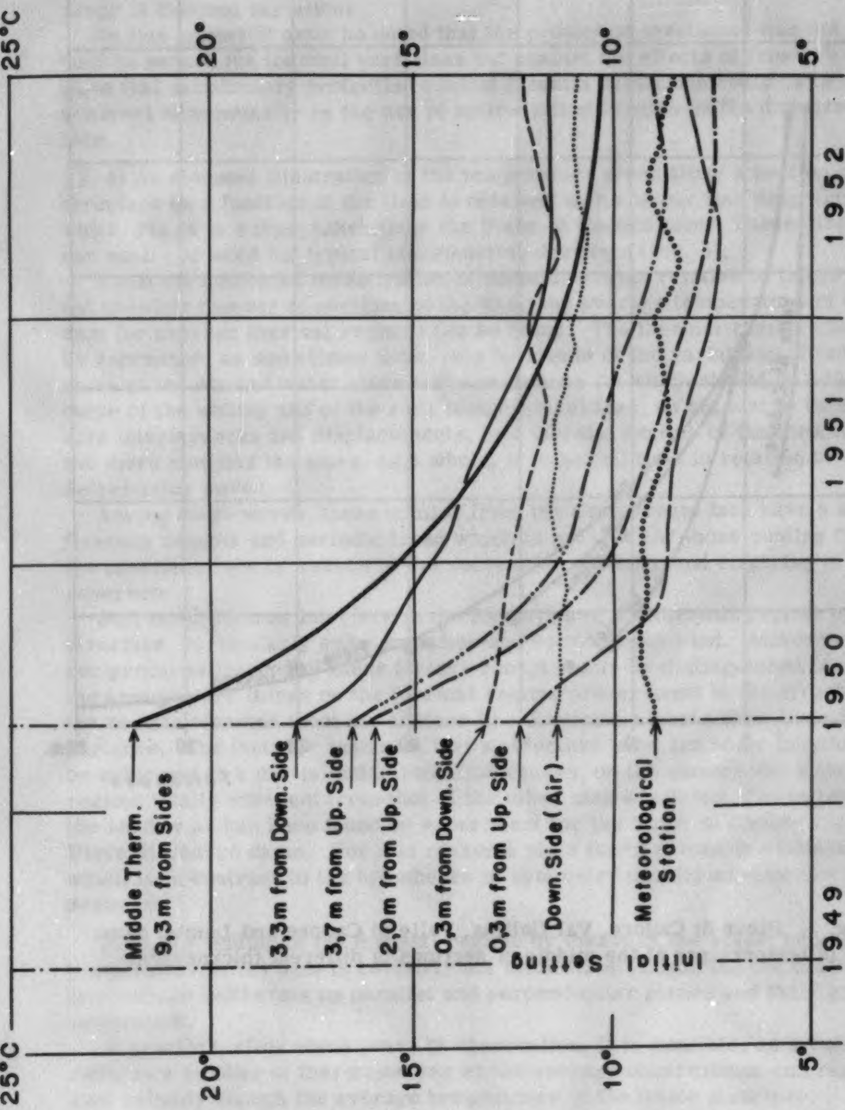

Fig. 2. Pieve di Cadore dam: mobile annual averages of temperatures in a section of a thickness of 18.6 m (exhaustion of setting heat).

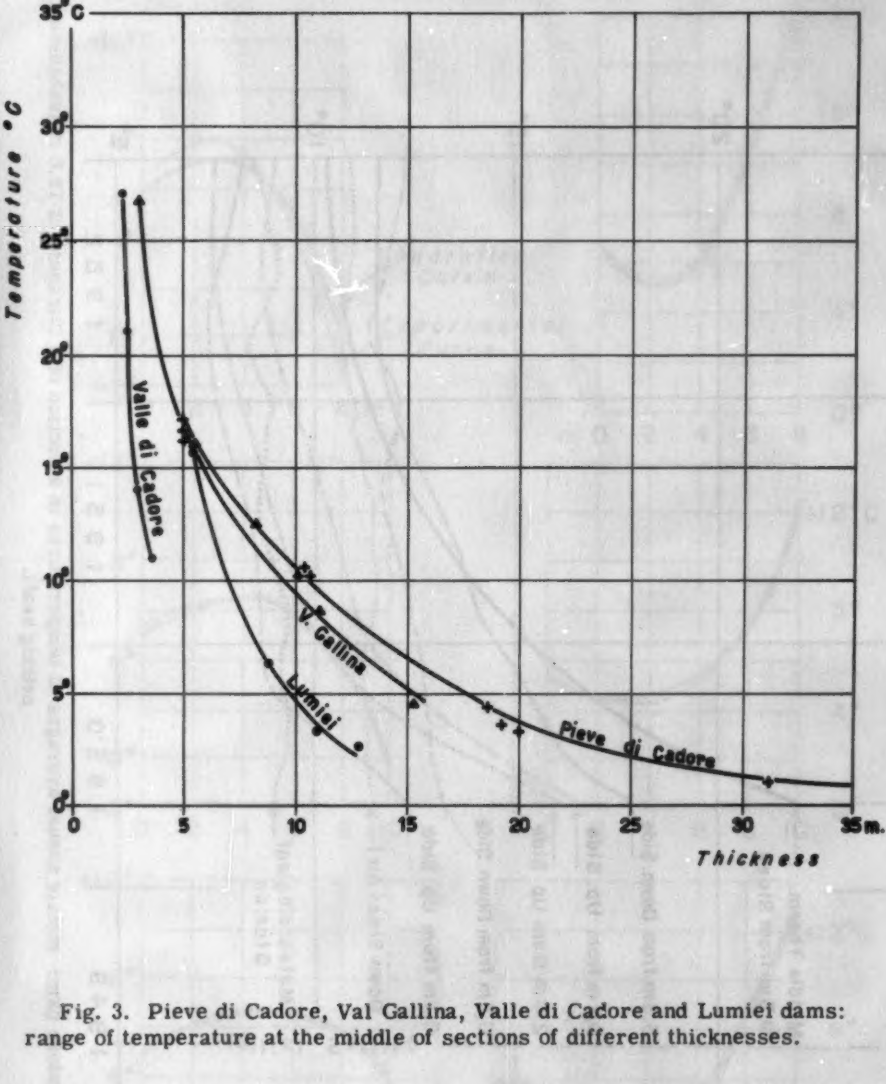

Fig. 3. Pieve di Cadore, Val Gallina, Valle di Cadore and Lumiei dams: range of temperature at the middle of sections of different thicknesses.

Finally, notably lower figures are found for the Valle di Cadore dam which has practically the same technological and climatic features as the Pieve di Cadore dam, but where the structure is protected upstream and downstream by concrete block facings of 0.30 m thickness, which clearly reduces the range of thermal variations.

On this matter it must be noted that the protection mentioned was not built to reduce the thermal variations but against the effects of frost. We believe that satisfactory protection against thermal variations could rather be achieved economically by the use of antiradiation vernish on the downstream face.

2. 4) An eloquent illustration of the temperature trend along a section of the structure as a function of the time is obtained with contour line diagrams of which Fig. 4 is a type, taken from the Pieve di Cadore dam. These diagrams can easily be used for typical stereometric drawings (Fig. 5).

From the combined investigation of these diagrams relative to the greatest possible number of sections of the dam, the average temperature of the dam (or average thermal regime) can be found. The thermal regime cannot be expressed, as sometimes done, only by means of the variations of temperature of the air and water since these variations (to which should be added those of the setting and of the rock temperature) lead, on account of successive interferences and displacements, to a thermal regime of the structure the more complex the more, as a whole, it is behind time in relation to the determining waves.

Among these waves, those coming from the downstream face have a sufficiently regular and periodic trend which is not true for those coming from the upstream face by reason of the succession of filling and emptying of the reservoir.

Still other factors interfere in the formation of the thermal regime of the structure, particularly solar radiation and action of the wind. Although the reciprocal influences of these factors cannot easily be distinguished (depending among other things on the thermal regime pre-existent in the structure), the results obtained from the contour line diagrams clearly show their importance. The fact, for instance, that a structure may, for some months even, be subjected to a partial solar radiation causes, on the sunny side, a thermal regime totally different from that of the other side which has always been in the shadow as has been found by experiment for the Valle di Cadore and Pieve di Cadore dams. For this reason a skew thermal load is established which is in contrast to the hypotheses of symmetry usually assumed by the designer.

2. 5) The finding of the thermal regime by means of the actual average temperature of the dam is certainly not immediate: requiring the drawing of fairly close isotherms on parallel and perpendicular planes and their graphic integration.

In practice, after some years of observation, it is possible, as a rule, to refer to a number of thermometers whose average observations can represent reliably enough the average temperature of the whole structure.

As an example, in fig. 6 diagrams of the average temperatures of the Pieve di Cadore, Valle, Val Gallina and Lumiei dams are shown compared with the average temperatures of the air (at intervals of ten days): these diagrams point out the differences mentioned before.

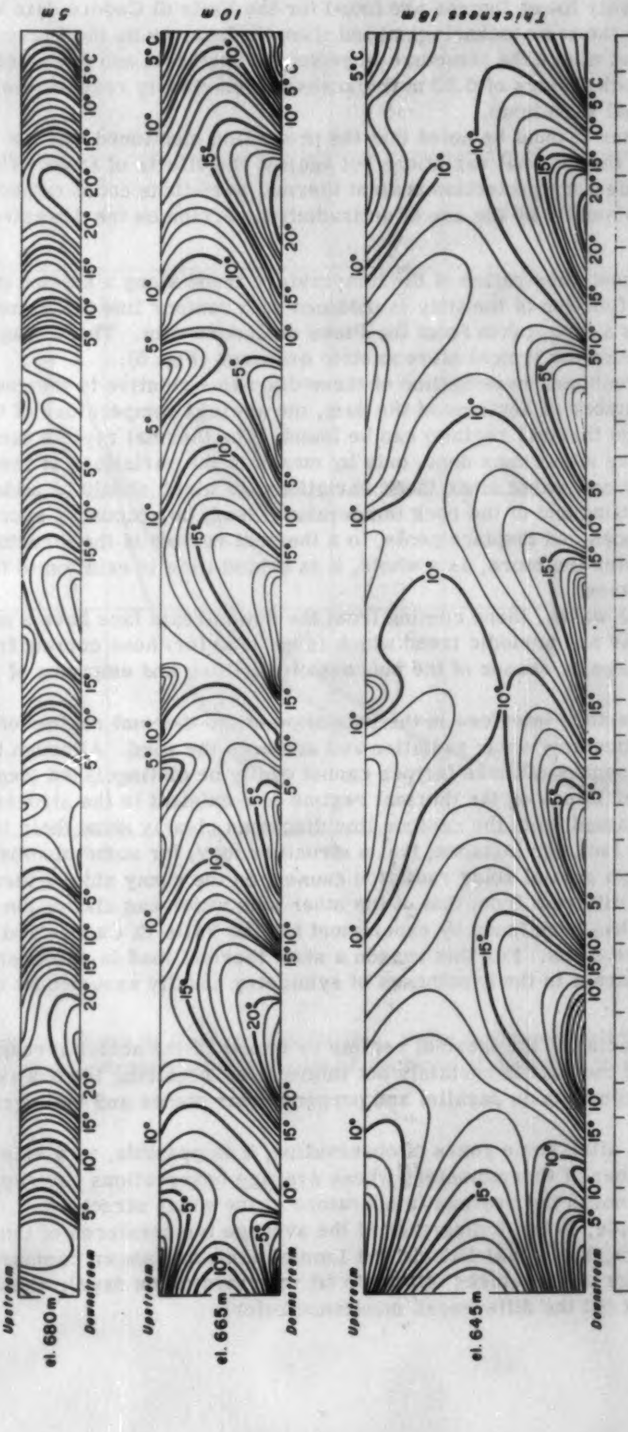

Fig. 4. Pieve di Cadore dam: distribution of temperature in relation to time and thickness, at different elevations.

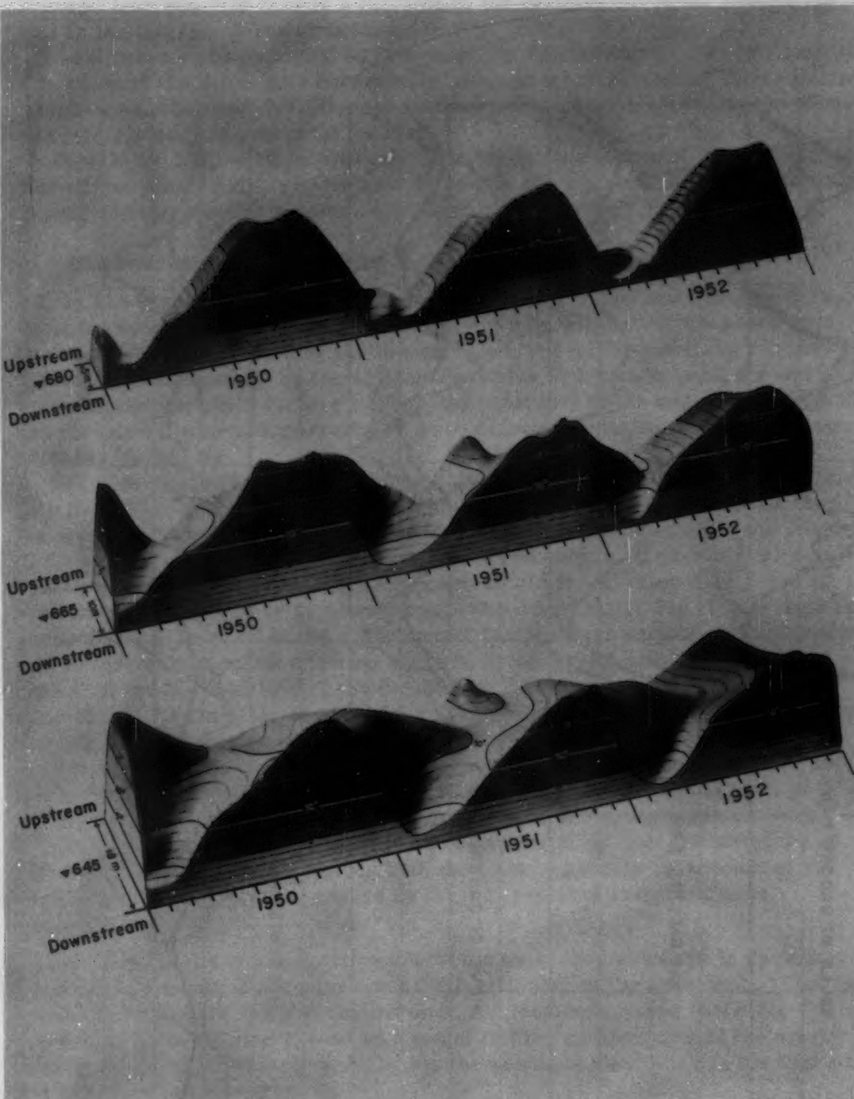

Fig. 5. Pieve di Cadore dam: stereometric representation of the distribution of temperature in relation to time and thickness at different elevations.

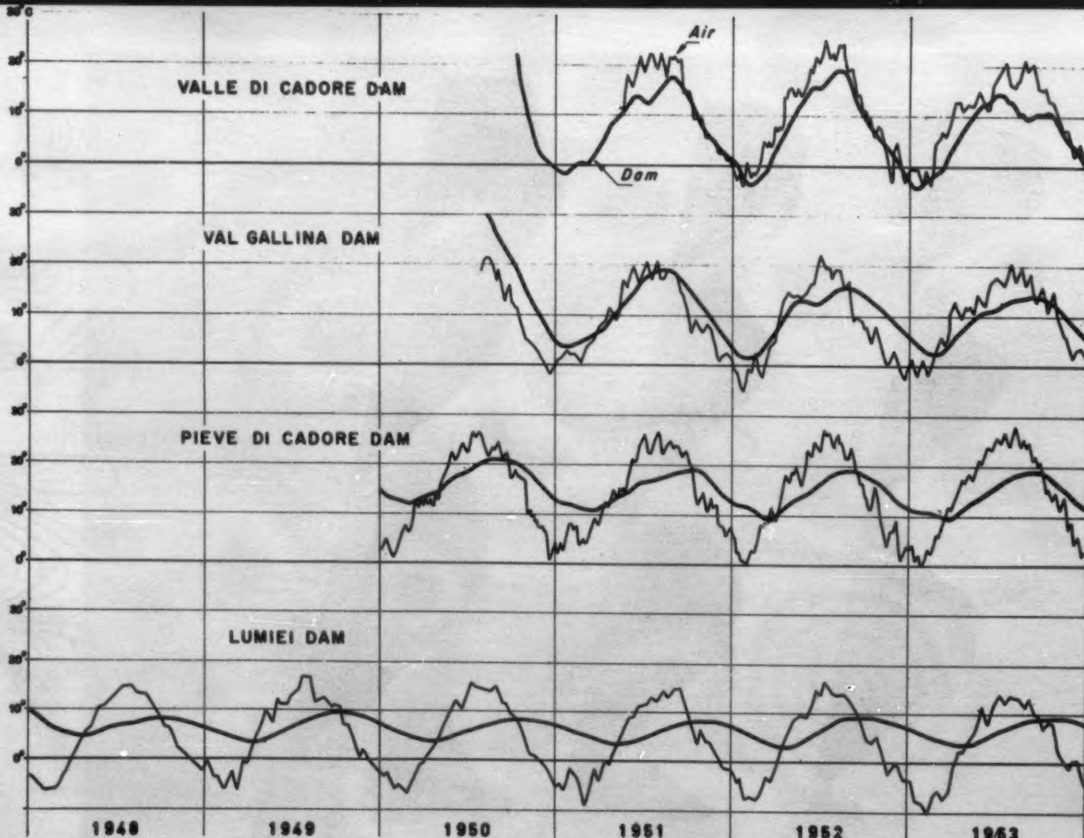

Fig. 6. Pieve di Cadore, Val Gallina, Valle di Cadore and Lumiei dams: average temperature of the dams.

But to know the thermal regime of a dam is not enough to calculate the thermal load which in turn is a function of the distribution of the regime itself in the interior of the structure.

Good results are obtained, on this point, by expressing the thermal load as a function of the difference between the average temperatures of the crest and foundation arches and the difference between the average temperatures of the downstream and the upstream faces.

Even if we confine these differences to the temperatures of a given section (the crown one), fairly reliable results can be quickly obtained, when the dam is subjected to a uniform solar radiation and ventilation.

3) - Measurements of Deflections

3. 1) The correlations between the variations in the thermal load and deflections in the structure are not immediate, not only because of the above mentioned difficulties of defining the thermal load, but more especially for the continuous interference by the actions due to the hydrostatic load, the evolution with time of other factors (creep and plasticity of the concrete and rock) and the quite frequent occurrence of non-periodic events such as microseisms, seiches, waves, etc.

Yet research with inclinographs has clearly shown a rotation of the structure due to the action of the daily thermal wave which is known to penetrate the mass of the concrete for 0.40 ÷ 0.60 m. This rotation which in the Pieve di Cadore dam reaches a maximum of $10^s \div 15^s$ occurs on days of high solar radiation, and is therefore at its minimum on cloudy or rainy days, even if the range of temperature is pronounced. The diagrams of Fig. 7 illustrate the phenomenon: the greater the difference of temperature between the upstream and downstream faces (a difference which, as observed, may be considered typical of the thermal load of the structure), the more this phenomenon is accentuated. A time-lag of about four hours between the maximum difference and the maximum deflection has been noted.

The diagram also indicates a progressive rotation towards upstream, so that at the end of each daily cycle the point does not return to the point of departure: this is due to the continuous increase of the temperature load.

The observations have brought to light another fact: that the influence of the daily thermal wave decreases with the time: probably on account of the hardening of the structure, caused by the increment of the modulus of elasticity.

3. 2) The quantity of the rotation due to the daily thermal wave is obviously linked with the type of structure and the functioning of the reservoir. The rotations recorded for the Val Gallina dam, for instance, where there are marked daily variations of load on account of the requirements of the nearby power station of Soverzene, are clearly the result of variations in the thermal and hydrostatic loads (Fig. 8).

Now by analyzing brief periods of rapid filling and emptying of the reservoir during which only slight variations in temperature are verified, it has been possible to determine an approximately linear relation between loads and rotations (from elevation 632 m to 642 m the rotation is of 6^s 6 for 10 m of hydrostatic load).

Using this relation, the daily global rotation has been depurated of the quota due to the hydrostatic load, thus obtaining a residual diagram, which should express the rotations exclusively due to the thermal load: this occurs fairly satisfactorily with a rotation of 1^s 7 for each 10^o C of thermal load.

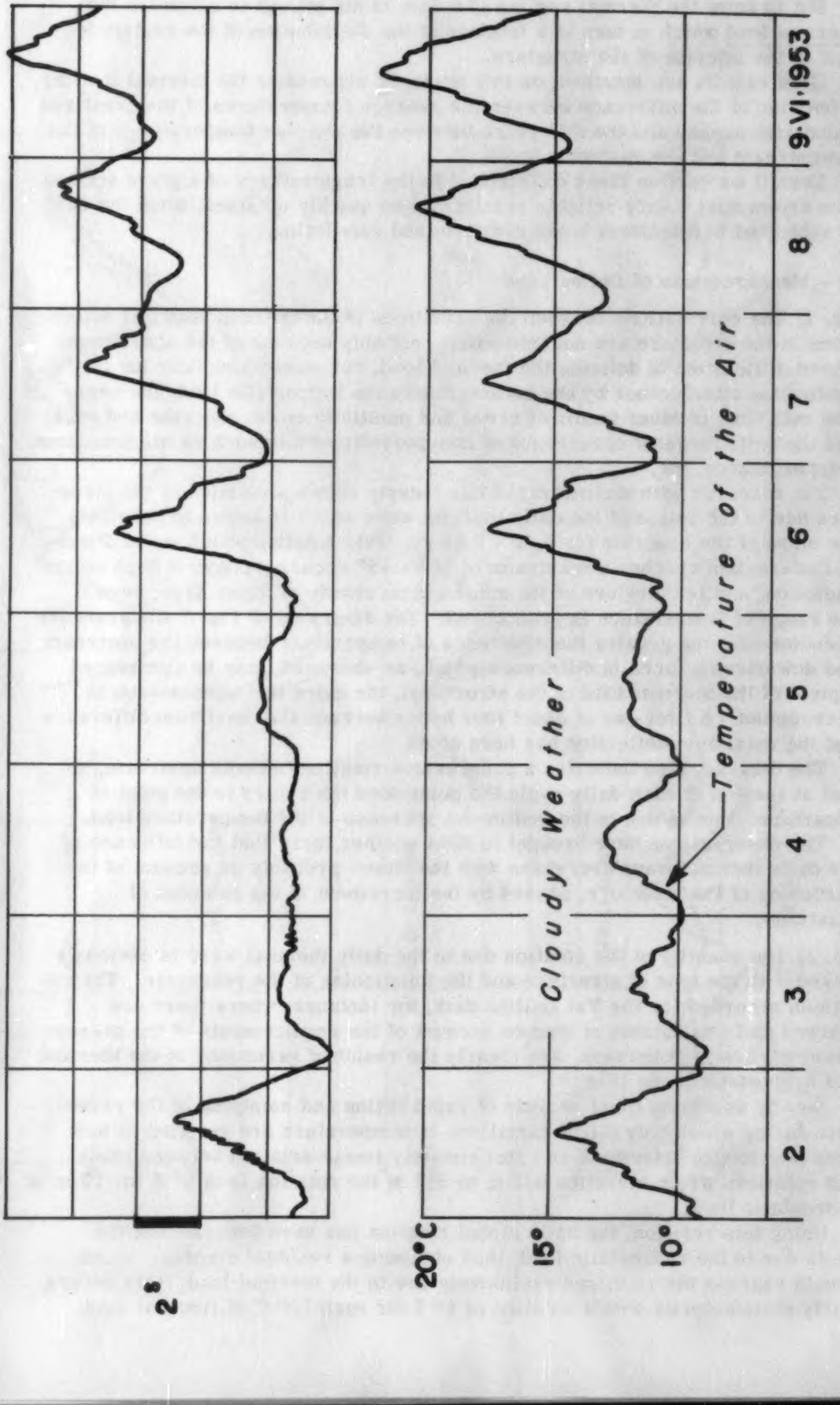

Fig. 7. Pieve di Cadore dam: daily variations of temperature and correlative rotations (Block XIV at elevation 624).

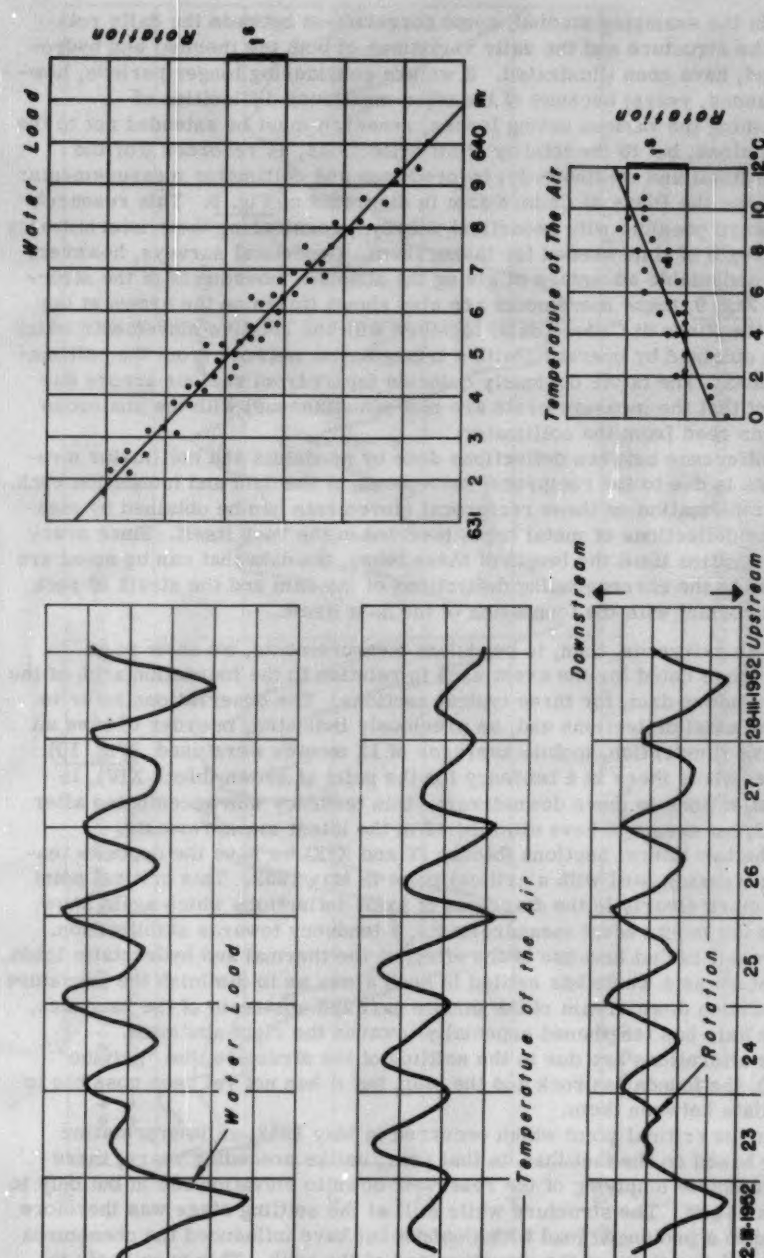

Fig. 8. Val Gallina dam: correlations between rotations and thermal and hydrostatic loads.

3. 3) In the examples studied, some correlations between the daily rotations of the structure and the daily variations of both the thermal and hydrostatic load, have been illustrated. If we are considering longer periods, however (seasons, years) because of the often mentioned difficulties of distinguishing the various acting forces, research must be extended not to the rotations alone, but to the totality of all deflections, as recorded (for the structure itself and continuously) by pendulum and collimator measurements: as shown for the Pieve di Cadore dam in diagrams of Fig. 9. This research is not always possible with geodetical surveys, considering their intermittency and the length of time needed for taking them. Geodetical surveys, however, have the undeniable advantage of giving the absolute movements of the structure. In Fig. 9, these movements are also shown (taken on the crown at the crest of the Pieve di Cadore dam) together with the relative movements which would be obtained by operating with a triangulation network from the collimation stations. The latter obviously coincide (apart from various errors due to the fact that the measurements are non-simultaneous) with the analogous deflections read from the collimator.

The difference between deflections done by pendulum and collimator measurements is due to the reciprocal movements of the dam and foundation rock. Further information on these reciprocal movements can be obtained by measuring the deflections of metal tubes inserted in the rock itself. Since practical necessities limit the length of these tubes, the data that can be noted are restricted to the corresponding deflections of the dam and the strata of rock in direct contact with the foundation of the dam itself.

3. 4) With reference, then, to pendulum measurements, we show as an example those noted for the crest arch in relation to the foundation arch of the Pieve di Cadore dam, for three typical sections. The observations refer to radial and axial deflections and, as previously indicated, in order to have an expressive illustration, mobile averages of 12 months were used (Fig. 10).

On the whole there is a tendency for the point at crown (block XIV), in radial deflections to move downstream: this tendency was accentuated after May 1952, but seems to have diminished in the latest measurements.

For the two lateral sections (blocks IV and XIX) we have the opposite tendency (upstream), still with a critical point in May 1952. This critical point appears more clearly in the diagrams of axial deflections which again show, though in the most recent measurements, a tendency towards stabilization.

The result is that because of the effect of the thermal and hydrostatic loads, the structure as a whole has settled in such a way as to diminish the curvature by a deflection downstream of the middle part and upstream of the haunches, while the axis has lengthened especially towards the right abutment.

These alterations are due to the settling of the structure, the "pulvino" (cushion), the foundation rock and the plug, but it has not yet been possible to differentiate between them.

As for the critical point which occurred in May 1952, an interpretation might be based on the fact that, in that year, unlike preceding years, there was no complete emptying of the reservoir down to elevation 630 m but only to elevation 644 m. The structure while still at the settling stage was therefore subjected to a prolonged load which cannot but have influenced the phenomena of creep and plasticity of the structure and of the rock. This hypothesis is partly confirmed by the fact that the measurements of the modulus of elasticity of the rock showed a diminution of the modulus from 1949, prior to the first

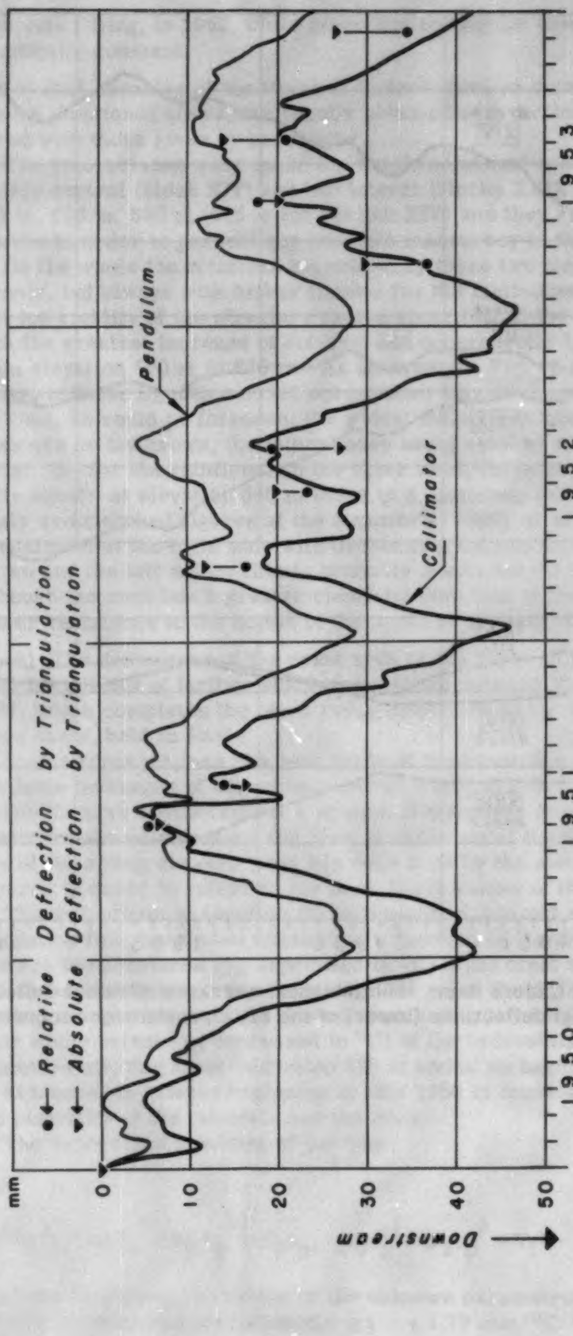

Fig. 9. Pieve di Cadore dam: deflections of the crown point of the crest arch according to the pendulum and the collimator.

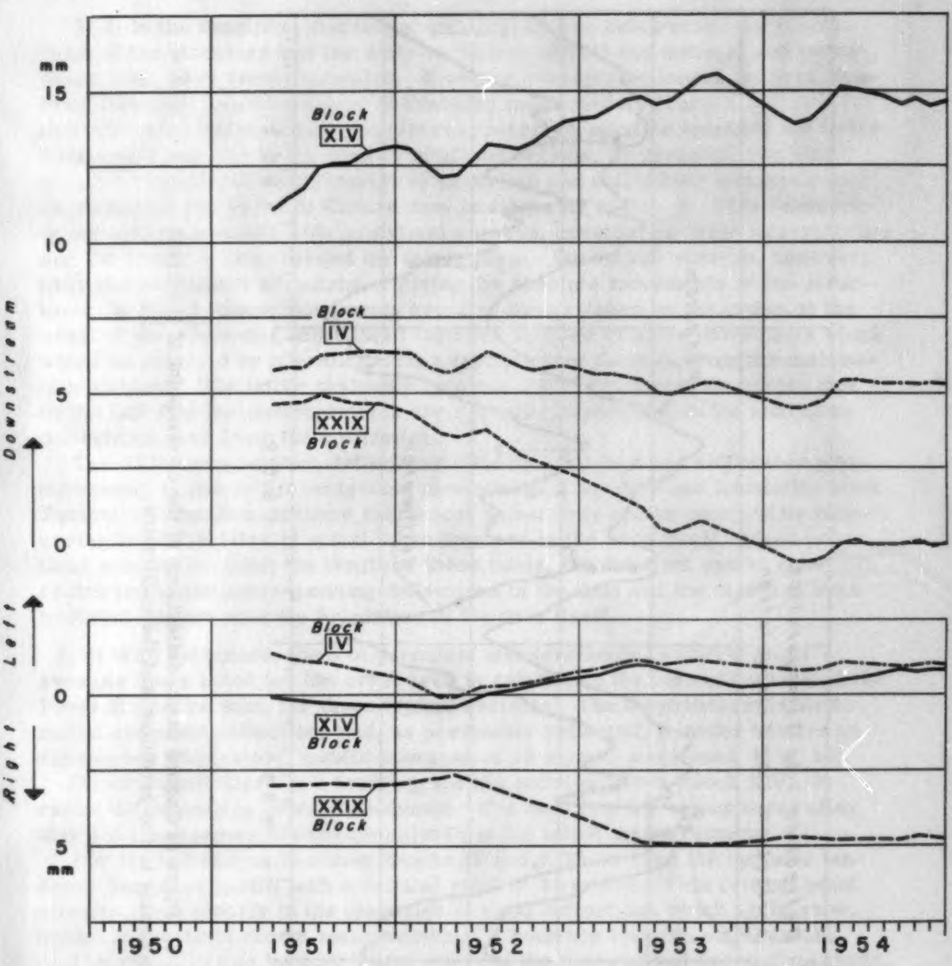

Fig. 10. Pieve di Cadore dam: mobile annual averages of radial deflections (upper) and axial deflections (lower) of the crown cantilever according to the pendulum.

complete filling, to 1952, while since this period the modulus has remained practically constant.

3. 5) Still speaking of the Pieve di Cadore dam, on completion of the research mentioned above, the results obtained with inclinometers were compared with those given by pendulums.

The comparisons were made for the three named sections, one approximately central (block XIV) and two lateral (blocks XXIX and IV) at elevations 680 m, 660 m, 630 m (625 m for section XIV) and they refer to average monthly figures in order to prevent any possible inaccuracy in the measurements.

On the whole the rotations expressed by these two methods correspond closely, but always with higher figures for the inclinometers, which indicates that the rigidity of the structure varies according to the height; the section with the greatest increase of rotation and consequently the least rigidity is from elevation 660 m to 680 m. As an example, Fig. 11 shows the average yearly range, in these figures carried out between May 1952 and December 1955.

Thus, as could be foreseen, the widest deflections marked by the pendulums are on the crown, the lateral ones being smaller and practically symmetrical. As for the rotations, on the other hand, those at the base are practically equal: at elevation 660 m there is a maximum on the crown, and lower fairly symmetrical figures at the abutments: lastly at elevation 680 m there is a maximum at the right side with decreasing figures as we move towards the crown and the left side. This is probably due to the fact that on the right side, although the rock has a greater elasticity than that of the left side, there is a lesser resistence to the action of the arch, on account of its morphology.

3. 6) The deflections of the crest arch of the Pieve di Cadore dam have been the subject of further study, carried out between May 1950 and April 1954, which completes the one already presented at the 5th Conference on large dams, held in Paris.

In this research, use has been made of the numerous experimental data available (averages of the measurements over periods of three days) introducing them as coefficients in a system of equations in which the unknown quantities should represent the laws of variation of the deflections as a function of the acting causes. This has been done by the method of the least squares in order to calculate the most likely values of the parameters.

The deflections in question (1) expressed in mm and referring to the foundation line, have been considered a function of the difference between the average temperatures (θ_a expressed in $^{\circ}$C) of the crest arch at elevation 680 m and of the foundation arch at elevation 624 m; of the difference between the average temperatures of the downstream and upstream faces considered in their whole extent (θ_p expressed in $^{\circ}$C) of the hydrostatic load (h) expressed in meters starting from elevation 630 m and of an asymptotic function of time (t) expressed in months beginning in May 1950 in correlation with the creep and plasticity of the concrete and the rock.

The expression resulting of the type

$$1 = \alpha_1\theta_a + \alpha_2\theta_a^2 + \alpha_3\theta_a^3 + \beta_1\theta_p + \beta_2\theta_p^2 + \beta_3\theta_p^3 + \gamma_1 h + \gamma_2 h^2 + \gamma_3 h^3 + \delta_1 e^{-\delta_2 t} + k$$

has been resolved: the values of the unknown parametro are practical applications proved to be the following: $\alpha_1 = +1{,}18$ mm/$^{\circ}$C

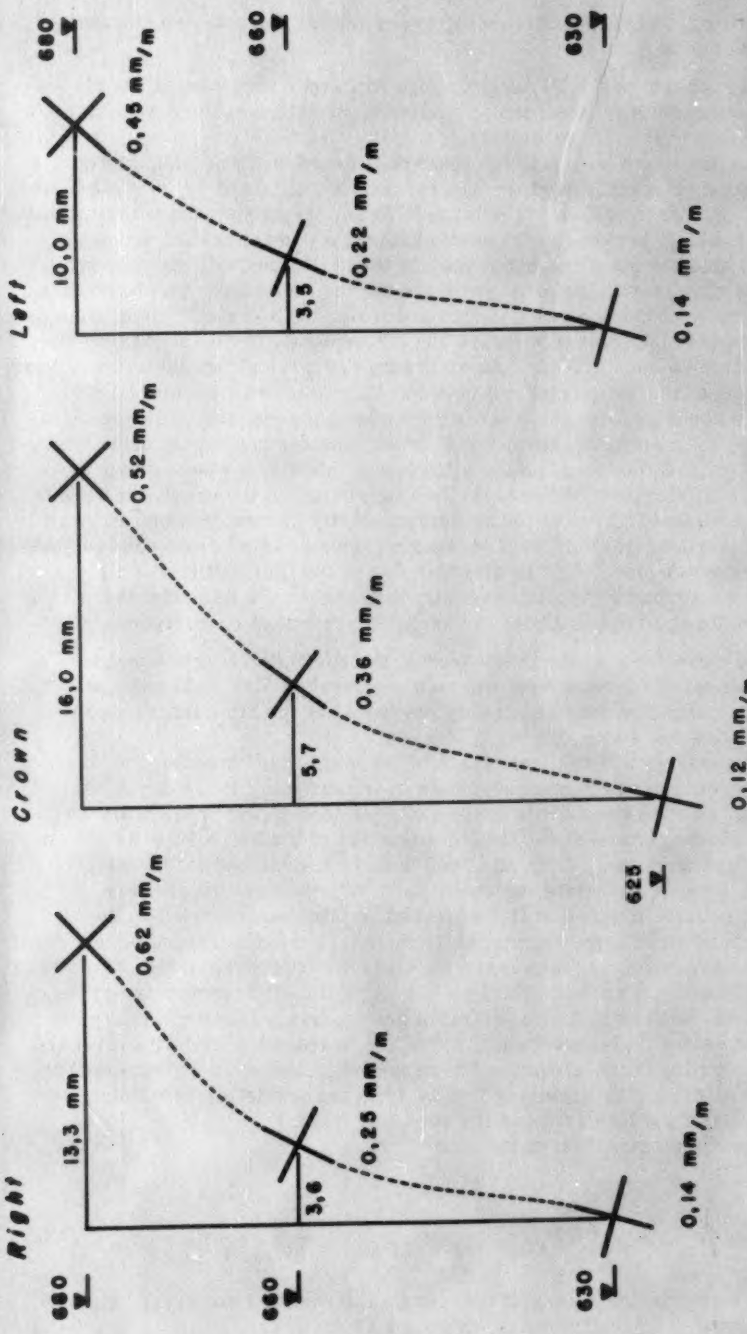

Fig. 11. Pieve di Cadore dam: average annual range of deflections and rotations for three cantilevers.

$\alpha_2 = +0{,}0073$ mm/°C $\quad \alpha_3 = -0{,}002983$ mm/°C $\quad \beta_1 = +0{,}43$ mm/°C
$\beta_2 = +0{,}0232$ mm/°C $\quad \beta_3 = -0{,}001479$ mm/°C $\quad \gamma_1 = -0{,}04$ mm/m
$\gamma_2 = -0{,}0012$ mm/m $\quad \gamma_3 = -0{,}000029$ mm/m $\quad \delta_1 = +7{,}16$ mm/month
$\delta_2 = 0{,}109$ mm/month $\quad k = +9.88$

The features of this research are summarised in Fig. 12.

Using the same data, for the deflections under examination, we have sought to classify the several effects according to their single causes, as they have been postulated. That is to say, we have verified the persistence of the loads by means of the result of single deflections for the time of their duration. In this elaboration clearly no account has been taken of the phenomena connected with time (creep, plasticity, etc.) since it is to be supposed that the external causes would every time affect a settled dam.

The following classification of the deflections results:

Quota due to the thermal load:

I) difference of temperature between the crest and foundation arches (α)	32 %
II) difference of temperature between downstream and upstream faces (β)	13 %
	45 %
Quota due to the hydrostatic load (γ)	45 %
Quota due to various unspecified causes and to errors	10 %
	100 %

The same research has been carried out for the Val Gallina dam also, obtaining satisfactory results, of the same type as those seen for the Pieve di Cadore dam.

3. 7) Geodetical surveys have been adopted also to check the movements of the upstream shores of the reservoir caused by successive fillings and emptying. There are 14 measurements for the Pieve di Cadore dam, for the first 8 of which there is a satisfactory correlation between the filling of the reservoir and the approach of the shores: this correlation, however, is not maintained in measurements taken after May 1952.

For the Val Gallina and Lumiei dams the measurements taken are not sufficient in number to define the phenomenon with precision; this is thus confirmed only for the first two years that the Pieve di Cadore dam was in operation.

On this matter, apart from the errors, not always mutually eliminating themselves which affect these measurements, it is considered that the thermal conditions of the rock (all the dams mentioned are built on limestone) and the thermal regime of the structure itself cannot but influence the movements of the shores: besides, the fixed stations of the triangulation network, however carefully constructed, are still anchored in the surface stratum of rock, particularly subject to thermal variations.

The results observed for the Pieve di Cadore dam would lead to the conclusion, already noted for other phenomena, that the rock, as the structure, is subject to movements of a certain significance especially in the first cycle of the working of the reservoir; subsequently a settling takes place to which,

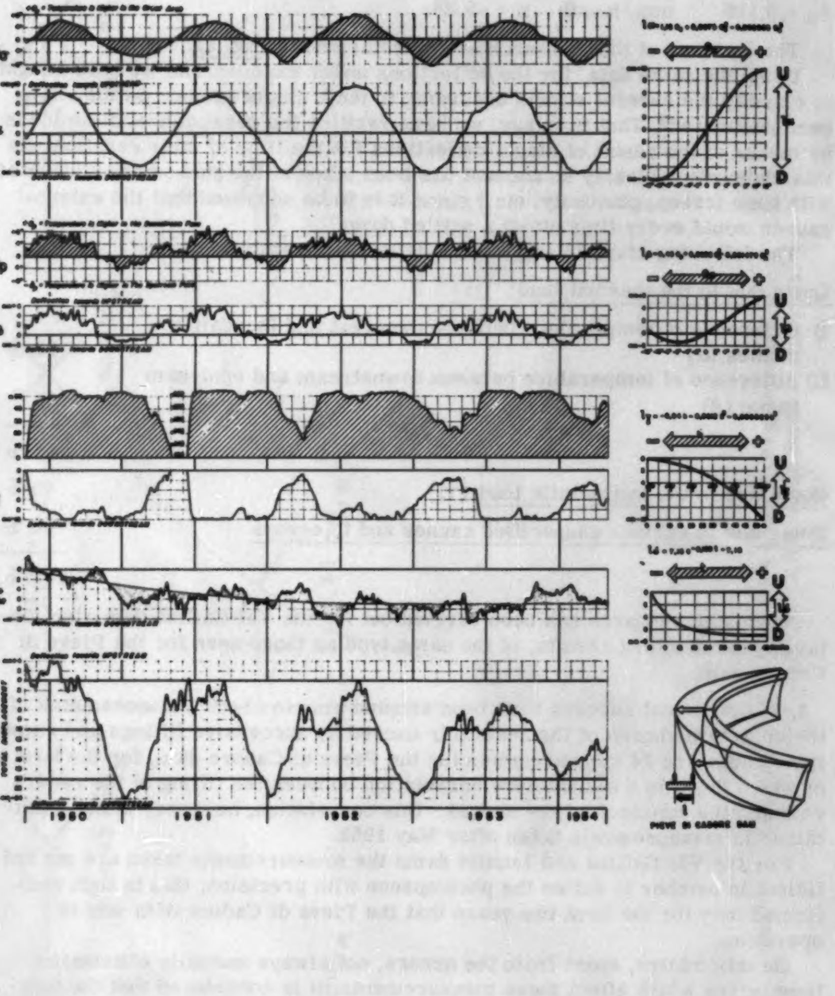

Fig. 12. Pieve di Cadore dam: analysis of deflections of crest arch.

in this specific case, in some degree corresponds the uniform imbibition of the rocks. But the problem is too closely connected with geological and morphological characteristics and those of the cycle of the reservoir to be possible to make for it any generalizations.

From what has been said, however, the great importance of research on the actions and reactions of the rock is made evident: research which up to date has perhaps been subordinated to that on the structure in itself.

4) - Measurement of Deformations

4. 1) Numerous data relating to deformations are recorded, for the structures under examination, by acoustical and resistance strain-meters either single or in rosettes. The strain-meters are generally laid in the interior of the dam at $0{,}60 \div 1{,}00$ m from sides to eliminate the troubled external zone.

But the elaboration from the elastic deformations to stresses, as we know, is not immediate because of the actual lack of information on the accuracy of the values of variations of the modulus of elasticity.

This modulus increases with time, but this increase generally is opposed and overcome by the phenomena of plasticity and creep due to the action of the loads.

For a structure subjected to continuous or periodical loads it is convenient to adopt a conventional modulus based on the behavior of the structure itself to the total of the various actions of elasticity, plasticity and creep.

Various methods have been suggested for finding this modulus for the structure as a whole (jacks, residual tensions etc.) since results from tests in the laboratory have no meaning in this field of research.

Besides the seismic method which will be mentioned later, research on tests loaded accorded to Prof. Oberti's method has been chosen, followed here with some variants: these are prismatic tests of about 2 x 0,2 x 0,2 m merely supported at the ends and loaded, including their own weight to get a stress of 15 and 25 kg/cm^2.

The load is applied 28 days from the setting and the deformations are recorded periodically on the upper and lower faces by means of removable strain-meters.

The diagrams of Fig. 13 show the variation of the deformation for elasticity, plasticity and creep due mainly to the load (half the difference of the upper and lower sides deformations) and the variation of the deformation principally due to actions of shrinkage, thermal expansion etc. (half the sum of the upper and lower sides deformations).

From the study of these deformations it appears that the initial modulus, of 260 000 kg/cm^2, is reduced, after about 40 months of load to 145 000 kg/cm^2 for the tests loaded at 25 kg/cm^2 and to 165 000 kg/cm^2 for those loaded at 15 kg/cm^2.

It is interesting to observe how the ratios of creep (ratio between the initial modulus of elasticity and the final conventional modulus) is a little less than 2: a relatively low figure compared with those met in other experiments of the kind.

The data just examined referred to tests of concrete used for the Val Gallina dam: experiments of the same type have been made also for the Pieve di Cadore dam which confirm the previous results. But even these figures of the moduluses calculated on tests in particular conditions of setting and load cannot give an idea of the true average modulus of the whole structure, i.e., the only element that can represent the resistance of the structure

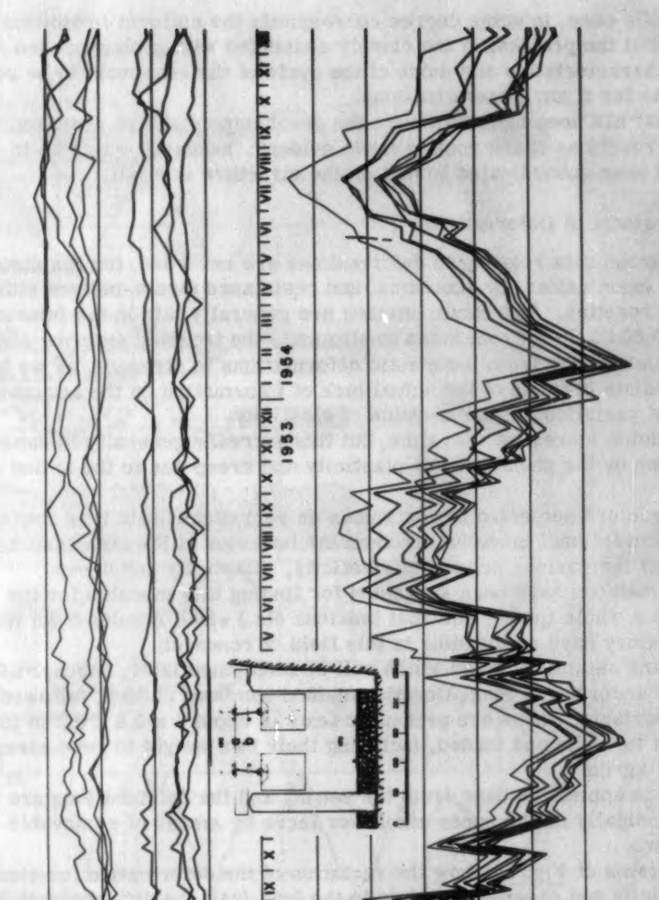

Fig. 13. Val Gallina dam: deflections in relation to time of upper and lower side of prismatic tests.

itself and that can be obtained on the contrary from tests on models. For this it would be necessary that the ratio between modulus of the tests from material of model and average total modulus of the model would have to be equal to the ratio between modulus of the test from material of structure and average total modulus of the structure.

5) - Geophysic Measurements

5. 1) Interesting information on the modulus of elasticity of the structure and of the rock can easily be obtained by seismic methods by which we can readily determine a dynamic modulus of elasticity, more or less closely correlated with the static modulus of elasticity with which we usually deal. Whatever method is used to discover the dynamic modulus it is always a question of rapid and economical processes which afford a truly statistical value of the modulus which are of interest. This also affords comparisons useful for a preliminary evaluation: for rocks of the limestone type, for example, Mr. Semenza has proposed a system of comparative evaluations of the elastic properties of the rock (middling rock up to $2{,}5.10^5$ kg/cm^2; good rock from 2,5 to 5.10^6 kg/cm^2; very good rock from 5 to $7.5.10^5$ kg/cm^2; excellent rock 7,5 to 10.10 kg/cm^2) which, taking into account the construction experience on various types of rock, has given more than satisfactory results.

It has been remarked that to express the dynamic modulus in kg/cm^2 may cause some confusion in regard to the static modulus which is traditionally expressed in the same units. Far from being impossible, it is advisable to refer the elastic properties of the rock calculated by the seismic method, to the velocity of propagation of the elastic waves expressed in m/s. In fact, modulus of elasticity or velocity of propagation represent conventions to express in a single figure the elastic qualities of the material which interest the constructor: between two conventions it is better to refer to the most convenient.

5. 2) By means of the dynamic modulus it is also possible to observe easily the anisotropy of the foundation rock, and so take due account of this in the design of the dam, and also to have an idea of the decline of the elastic properties of the material; a decline due to that set of phenomena of settling repeatedly mentioned.

As an example, experiments carried out with the dynamic method on the rock which forms the right abutment of the Pieve di Cadore dam, in 1948 to 1949 before and during its construction, led to the finding of an average modulus of elasticity of 480 000 kg/cm^2. When the experiments were repeated in 1952 and 1953, three and four years respectively after the first filling of the reservoir, the average modulus was found to be 330 000 kg/cm^2, a decrease of 150 000 kg/cm^2 compared with the original figure. In 1953 other experiments of the same kind were repeated at the Lumiei dam, built in 1947. Here, while downstream from the dam the modulus was about 1 000 000 kg/cm^2, upstream the figure was about 800 000 kg/cm^2. In this case, too, there was a decline on elastic properties in the area affected by the waters, although somewhat less than that observed at Pieve di Cadore.

According to Prof. Caloi who has carried out this research the decline of the modulus may be caused by an increase in the porosity of the rock, determined by the stress due to the increased pressures and their more or less sharp variations.

Seismographic observations in fact enable to conclude that in the rock which forms the bed of a reservoir especially near the foundations, from the beginning of the work to its completion and the first filling of the reservoir, and to a lesser degree in subsequent fillings, a continuous settling has been in progress, determined by the disruption of the equilibrium previously existing between the forces in the medium. The upsetting of this equilibrium may manifest itself in true small earthquakes, at least in myriad of small strokes affecting the rock surrounding the dam. This action of shocks if continued for years, causes minute fractures in the rock and innumerable small lesions which tend to increase its porosity in a wide sense of the word. This minute, wearing action is manifested particularly in the whole of the rocks supporting the dam, especially in arch dams.

We note that this interpretation might be used to explain the diversity of elastic properties found in rocks of the same geological period and the same chemical composition: a diversity which would be due to the different stresses which the various rocks have undergone during the ages.

5. 3) By geophysical means (inclinographs, seismographs, vibrometers, etc.), aspects of the behaviour of the structure and foundations can be examined which would escape other kinds of investigation.

Thus with the combined use of seismographs and inclinographs it has been possible to establish the fact that the Pieve di Cadore dam belongs to a single geodetic block. At present the geophysicists have reached the conclusion that, especially in the areas subjected to earthquakes, the surface stratum of the earth crust is made up of relatively small blocks (horizontal dimensions of the order of 7 + 14 kilometers with analogous thicknesses) bounded by recent faults or by surfaces of fractures liable as a mass to mutual sliding without perceptible deformation. Now it is very important that a dam, especially an arch dam, should be based on a single geodetic block; research of the kind is therefore necessary above all in the preliminary stage of study and planning.

5. 4) Prof. Caloi has also observed that an important correlation exists between a preliminary abnormal clinographic activity and a subsequent abnormal macro and micro-seismic activity. This would indicate an intimate dependence between the gradual movements of strata of the rocks, under the influence of tensions acting in them, and the sudden destructions of equilibrium (earthquakes) which these tensions may ultimately provoke.

As an example, in the diagram of Fig. 14 it is shown, according to their components, the variations of rotation which, since the 22nd September 1954 have gradually started in the foundation rock of the Ambiesta dam (under construction). The rotation is first in a northerly, then in a decidedly westerly direction. The rotation in this direction was shown to be particularly active from October 3rd to 8th, a period in which the first very slight instrumental shocks occurred, gradually increasing in frequency, confirming the tension present in the areas in slow movement. Subsequently the rotations again took a northerly direction; corresponding to this change of rotation, on October 11th a seismic shock of the intensity of 10^{18} erg occurred, followed by an almost uninterrupted series of small instrumental shocks with the task of exhausting the residual tensions in the stratifications affected by the earthquake.

6) - Conclusions

6. 1) The observations and results given here are intended to show the

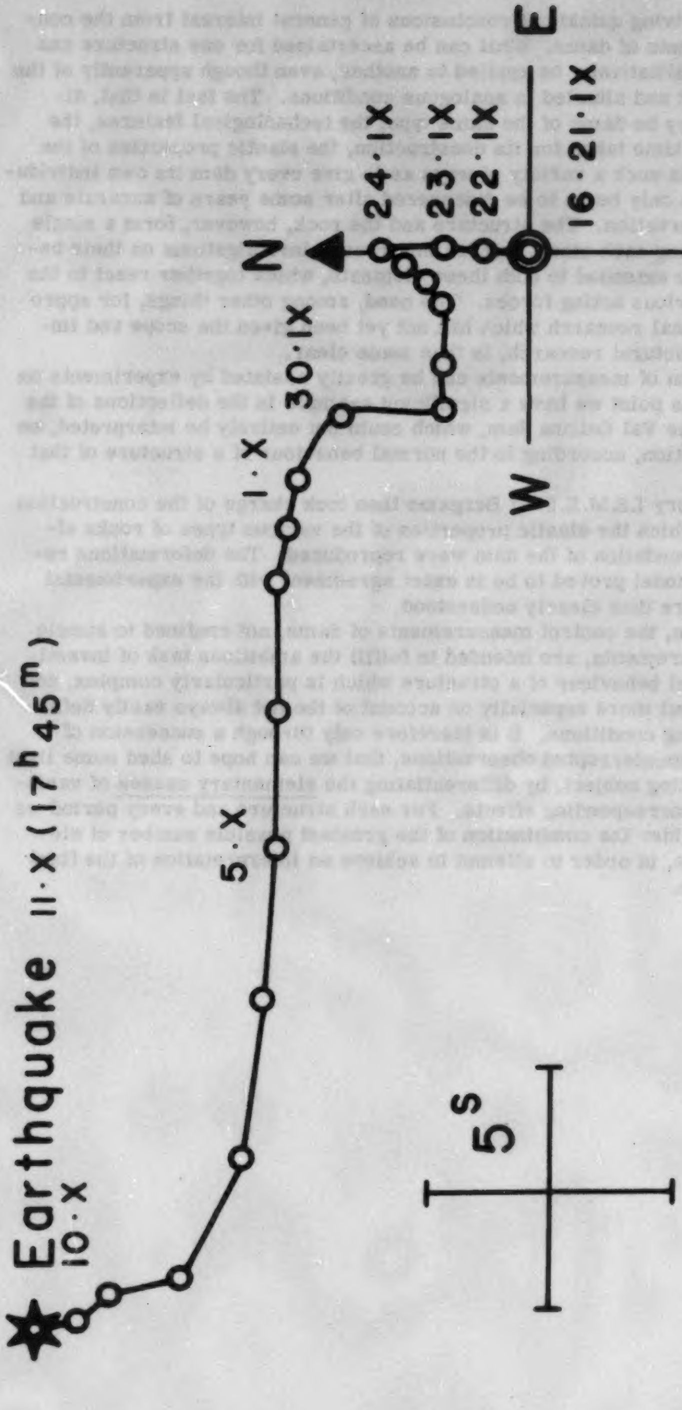

Fig. 14. Ambiesta dam: inclinographic activity observed near to the rock of foundation.

difficulty of arriving quickly at conclusions of general interest from the control measurements of dams. What can be ascertained for one structure can hardly, even qualitatively, be applied to another, even though apparently of the same type, built and situated in analogous conditions. The fact is that, although there may be dams of the same type, the technological features, the means, and the time taken for its construction, the elastic properties of the rock interfere in such a variety of ways as to give every dam its own individuality, which can only begin to be discovered after some years of accurate and continuous observation. The structure and the rock, however, form a single whole, influencing each other, and for this reason investigations on their behaviour must be extended to both these elements, which together react to the effect of the various acting forces. The need, among other things, for appropriate geophysical research which has not yet been given the scope and importance of structural research, is thus made clear.

Interpretation of measurements can be greatly assisted by experiments on models. On this point we have a significant example in the deflections of the crest arch of the Val Gallina dam, which could not entirely be interpreted, on a first examination, according to the normal behaviour of a structure of that type.

The laboratory I.S.M.E.S. of Bergamo then took charge of the construction of a model in which the elastic properties of the various types of rocks affected by the foundation of the dam were reproduced. The deformations recorded in the model proved to be in exact agreement with the experimental ones, which were thus clearly understood.

In conclusion, the control measurements of dams, not confined to simple "safety" measurements, are intended to fulfill the ambitious task of investigating the actual behaviour of a structure which is particularly complex, not only in itself, but more especially on account of the not always easily definable surrounding conditions. It is therefore only through a succession of numerous and uninterrupted observations, that we can hope to shed some light on this interesting subject, by differentiating the elementary causes of variation from the corresponding effects. For each structure and every period we must then consider the combination of the greatest possible number of elementary causes, in order to attempt to achieve an interpretation of the final resulting whole.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

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JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 (HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

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JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023 (ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

c. Discussion of several papers, grouped by Divisions.